

**DEVELOPMENT AND ASSESSMENT OF A CONTINUOUS
SIMULATION MODELLING SYSTEM FOR DESIGN
FLOOD ESTIMATION**

**JC Smithers • KT Chetty • MS Frezghi •
DM Knoesen • MH Tewolde**

WRC Report No. 1318/1/07



Water Research Commission



DEVELOPMENT AND ASSESSMENT OF A CONTINUOUS SIMULATION MODELLING SYSTEM FOR DESIGN FLOOD ESTIMATION

Report to the
Water Research Commission

by

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WRC Report No 1318/1/07

ISBN 978-1-77005-535-3

May 2007

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EXECUTIVE SUMMARY

Background

The estimation of design floods is necessary for the design of hydraulic structures. The under estimation of design floods will result in the failure of hydraulic structures with consequent economic losses and possible loss of life. The over design of hydraulic structures results in inefficient use of resources. The choice of an acceptable and cost-effective engineering solution is dependent upon having reliable estimates of the frequency of floods, both in terms of peak flows and volumes of water.

Standard techniques for flood estimation have been developed for most countries and procedures for design flood estimation may be broadly categorised as methods based on the analysis of observed floods and rainfall based methods. The situation which faces design engineers and hydrologists most frequently is when no, or inadequate, observed streamflow data are available at the site of interest and then either regional approaches or rainfall-runoff models have to be used for design flood estimation.

In the absence of adequate streamflow data, a number of rainfall event-based approaches are commonly used. In South Africa these include the Rational, unit hydrograph and SCS-SA methods. The procedures for applying the widely used unit hydrograph approach in South Africa were developed in the early 1970s and very little development or refinement has been made since then. Major shortcomings of most event-based methods include the implicit assumption that the exceedance probability of the simulated streamflow is the same as that of the input design rainfall and that antecedent soil moisture conditions prior to large storm events are not accounted for. The limitations of event-based approaches to design flood estimation can be overcome by adopting a continuous simulation approach to rainfall-runoff modelling where the major processes responsible for converting the input catchment rainfall into runoff are explicitly modelled. The use of a continuous water budget ensures that the antecedent conditions prior to each rainfall event are simulated, and the exceedance probability is computed directly from the simulated streamflow and not inferred from the rainfall.

Project Objectives

The objectives of this project were to develop a methodology, using a continuous simulation modelling approach, which could be used for flood frequency estimation in South Africa.

Methodology

The *ACRU* model was selected as the continuous simulation model as it operates on a daily time step, the simulated streamflow is sensitive to soils and land cover input and the model has been used to estimate design floods in previous pilot studies (Smithers *et al.*, 1997; Smithers *et al.*, 2001). A number of areas were identified in this project where the *ACRU* modelling system could be further developed or refined for design

flood estimation. These included:

- Investigations into the scale of application and levels of soils and land cover information required to apply the *ACRU* model for continuous simulation modelling in order to estimate design floods.
- The development and evaluation of a method to disaggregate daily rainfall into hourly totals in South Africa in order to improve the shape of simulated hydrographs and estimation of peak discharge.
- An assessment of procedures used to merge raingauge and radar data and the use of the merged rainfall fields in the development of a methodology to improve the estimation of rainfall falling over a catchment when using historical daily rainfall data.
- An assessment of the stochastic, fine resolution space-time String-of-Beads model to simulate long series of rainfall over a catchment which would enable long series of simulated streamflow to be generated in order to estimate design floods.
- The development and assessment of techniques for flood routing in ungauged catchments.
- In a parallel study, the development and application of adjustments for antecedent soil moisture conditions to Curve Numbers, used in the SCS design flood estimation model, was investigated.
- These developments and refinements to the *ACRU* modelling system made in this study were incorporated into the *ACRU* model and the Thukela Catchment was used to assess the use of continuous simulation modelling for design flood estimation.

Scales Issues Related to the Configuration of the *ACRU* Model for Design Flood Estimation

In the development of a Continuous Simulation Modelling (CSM) system in this study, the spatial resolution at which continuous simulation modelling is implemented is important in order to model the heterogeneous hydrological responses from catchments. It was therefore necessary to investigate the appropriate range of scales at which the CSM system should be applied to identify the appropriate scales of representation of input information. Soil and land cover information play an important role in a CSM as these are the prime regulators of a catchment's response to rainfall, and therefore directly influence the hydrological response of a catchment. The objective of this component of the study was to investigate the appropriate scale at which the CSM should be configured with respect to levels of spatial disaggregation of a catchment and of soil and land cover information required to give optimum results i.e. to investigate the optimal level of catchment discretisation for the *ACRU* model used in this study. Simulations at Quaternary Catchment (QC) scale and sub-quaternary scale (sub-QC) with different levels of soils and land cover information were undertaken in the Thukela Catchment.

The methodology adopted to perform this investigation required relatively un-impacted upstream sub-catchments within the Thukela Catchment which had adequate and reliable data. This limited the study as not many catchments are un-impacted and have long periods of flow records. Three QCs were selected with catchment areas ranging from 129 to 544 km². The majority of QCs within the Thukela catchment have catchment areas less than 500 km² and it was thus assumed that the results

obtained for the selected QCs would be valid for all QCs in the Thukela catchment. The results detailed in Section 3.2 indicate that:

- The larger QCs should not be modelled as lumped entities and should be discretised to hydrological response units (HRUs) as the best simulations were obtained using HRU scenarios.
- The use of area weighted and not modal soils information resulted in the best simulations.
- Modelling using more than one driver rainfall station per sub-QC yielded better results in QC 59 than when a single driver rainfall station was used, but the use of additional rainfall stations in QC 72 did not make much difference to simulated results. This could be attributed to the limited number of reliable representative rainfall stations in QC 72.

The recommendations from these results are that the optimum level of catchment discretisation in the Thukela catchment should include sub-quaternary catchments where appropriate, and all sub-catchments should be modelled using HRUs with area weighted soils information and, where possible, more than one driver rainfall station per QC should be used to represent the rainfall in the sub-catchments which constitute the QC.

Temporal Disaggregation of Daily Rainfall

The temporal distribution of rainfall, viz. the distribution of rainfall intensity during a storm, is an important factor affecting the timing and magnitude of peak flow from a catchment and hence the flood-generating potential of rainfall events. Rainfall disaggregation refers to estimating high temporal resolution rainfall values which can be aggregated to give values equal to observed coarser-scale totals.

Prior to this study, the *ACRU* model utilised four fixed regionalised synthetic temporal distributions to disaggregate daily rainfall. This suggests that a single distribution can be used to represent the temporal distribution of rainfall for a particular region. This, however, is not realistic and analysis of rainfall data shows that at a given location there are several temporal patterns ranging from nearly uniform rainfall to highly variable rainfall. Furthermore, the peak intensity can occur during any hour of the day, adding to the variability of temporal rainfall patterns. In order to account for the variability of temporal patterns of rainfall, a regionalised stochastic approach to daily rainfall disaggregation was developed in this study, as described in Section 3.3.

The daily rainfall disaggregation model developed is capable of producing 480 different temporal patterns with ranging levels of uniformity. The methodology was assessed independently at 15 sites in South Africa and both moments and statistics and extreme rainfall events were analysed. The results indicate that the disaggregation model reproduced the general distribution of rainfall relatively well, both when observed short duration data are available as well as in the absence of such information, but that the structure (sequencing) of the disaggregated rainfall requires additional refinements.

Spatial Distribution of Rainfall

Rainfall is highly variable in space and time and is the major driving hydrological force. Hence it is crucial to obtain an accurate estimate of rainfall in a catchment

when modelling the hydrological response from a catchment. Raingauges measure rainfall directly and rainfall depth accumulated over the period of interest is measured with a high degree of accuracy at points where the gauges are located. However, raingauge networks are too sparse to capture the spatial variability of rainfall. Radar, on the other hand, measures a volume-averaged reflected signal which is converted into rainfall and captures the spatial distribution of rainfall but needs to be calibrated using gauged rainfall. Merged rainfall fields, derived from radar and raingauges, are currently the best estimate of the "true" rainfall field in a catchment.

The objective of this component of the project was to improve the estimation of real and stochastic rainfall in a catchment. This was achieved by assessing a methodology to merge raingauge and radar data. Based on the premise that the merged rainfall fields are the best estimate of catchment rainfall, the short periods of available radar data were used to develop relationships between the merged rainfall fields for a catchment and data from a selected raingauge. This relationship can be used to adjust the much longer record of historical gauged rainfall data to better represent rainfall in the catchment. The String-of-Beads-Model (SBM) (Pegram and Clothier, 2002), which is a detailed space-time stochastic rainfall model, was also assessed for application to generate long sequences of rainfall over a catchment for use in a CSM. The Liebenbergsvlei Catchment was used as a study site as both radar and rainfall data from a dense network of gauges were available.

The merging technique was validated against data from tipping bucket raingauges used in the conditioning of the radar images. The conditional merging technique is intended to retain the rainfall depths used in the conditioning of the radar images in the merged rainfall field. The results obtained indicated that gauged rainfalls at the conditioning raingauges were not always retained, as the merging technique developed by Sinclair (2004) masks the area where the radar did not register any rainfall, even though raingauges in this area may have reported rainfall. This was done to avoid false rainfall in other parts of the area. When the masked values were removed from the comparison, a nearly perfect relationship was obtained between the conditioning raingauge data and the merged pixel rainfall values located at the conditioning raingauge. With the exception of the masked rainfall, the merging algorithm was determined to be correct.

The merging technique was independently verified using daily raingauges which were not used in the conditioning of the radar images. For most sub-catchments reasonably good verifications were obtained, with $R^2 > 0.70$. It was noted that the raingauges selected to represent the areal rainfall of the sub-catchments generally over-estimated the mean areal merged rainfall values of the sub-catchments by between 5% and 50%. The methodology developed can be used to provide improved estimates of average historical catchment rainfall for use in modelling and other hydrological studies, until such time as the period of record of the merged rainfall fields is adequate for hydrological studies.

The assessment of the SBM indicates that the model reproduced the observed statistics at a daily time scale reasonably well and better than at monthly or annual time scales. This result was not unexpected as the SBM is a short duration rainfall model designed to mimic rainfall values at a detailed temporal and spatial resolution and small errors at 5 minutes durations accumulate over longer durations to the errors evident at the daily and longer time scales. Spatially, the SBM reproduced the statistics of the selected raingauges used in the study. Therefore, it is concluded that

an appropriately calibrated SBM may be used in rainfall-runoff modelling which requires rainfall at detailed spatial and temporal resolutions and could be used as input to a CSM system in order to estimate design floods.

Flood Routing in Ungauged River Reaches

Hydrographs are lagged and attenuated as they flow down river reaches and through dams. In a CSM it is necessary to model the changes to hydrographs as they are routed through a catchment.

The Muskingum-Cunge method for flood routing is implemented in the *ACRU* model. In ungauged catchments, the user is required to specify the cross-sectional shape and dimensions of the reach, in addition to the slope and roughness coefficient for the reach. This input information is then used to estimate the depth : discharge relationship for the reach. In this study it was shown that the computed outflow hydrographs generated using the Muskingum-Cunge method, both with empirically estimated variables and variables estimated from cross-sections of the selected rivers, resulted in reasonably accurate computed outflow hydrographs with respect to peak discharge, timing of peak flow and volume. Hence, it is concluded that the Muskingum-Cunge method, with variables estimated using empirical relationships, can be applied to route floods in ungauged catchments in the Thukela Catchment and it is postulated that the method can be used to route floods in other ungauged rivers in South Africa.

Application of a Climate Classification System for Regional Adjustments of SCS Curve Numbers

The SCS method is a design event based approach to design flood estimation and has been adapted for use in South Africa (Schmidt and Schulze, 1987). The refinements to the SCS method for South Africa include the development of techniques to account for typical soil moisture status prior to large storms. Stormflow response is highly sensitive to a catchment's wetness. Many researchers have proposed procedures to adjust Curve Numbers (CNs) according to a soil water budget provide more realistic estimates of stormflow and peak discharge than when using only accumulated antecedent rainfall depths. However, the direct application of soil water budgeting procedures requires long and accurate daily rainfall and evaporation records, in order to estimate the change in soil moisture storage, ΔS , prior to runoff producing storm events. The problem then arises of how to estimate the values of ΔS in regions where only limited hydrological and very basic monthly climatological information is available.

The objective of this component of the study was to approach the soil water status of a catchment as a climatologically driven variable by assuming that changes to CNs due to Antecedent Soil Moisture (ASM) are similar internationally for similar climatic regions, and that climatic regions may be represented by a standard climate classification, such as the Köppen classification system, which was selected for use in this study. It was further hypothesised that within a specified Köppen Climate Class (KCC), ΔS was likely to be a function of the distribution of Mean Annual Precipitation (MAP) for major soil/vegetation combinations.

The concept of estimating ΔS from MAP for each KCC and specified soil/vegetation combinations was termed the *ACRU-Köppen* method. The approach is conceptually

sound, computationally simple, complies with hydrological fundamentals and does not move beyond the original intent of the SCS technique. A major strength of the approach is that it is a relatively simple technique, does not require a high level of expertise and is not data demanding for application. The use of the *ACRU*-Köppen method of CN adjustment was found to work well in a catchment in Eritrea and it is postulated that the method could be used in other data sparse catchments for the same KCC and similar soil/land cover characteristics to improve modelling of stormflow volumes and peak discharges from small catchments in developing regions, where adequate and accurate hydro-meteorological information are often not available.

Application of the CSM in the Thukela Catchment

The above developments and refinements were incorporated into the *ACRU* model and the Thukela Catchment was used as a case study. The application of the model proved challenging in an operational catchment where the observed data are not perfect, the network of daily raingauges is relatively sparse, transfers of water between catchments occur, irrigation is developed over time and other land cover changes occur.

Some challenges to modelling and design flood estimation which were noted during the project include the following:

- The problems associated with estimating rainfall over a catchment using a single raingauge were evident in some results when the simulated and observed streamflow did not correspond for some events. However, even though there may not be a one-to-one correspondence between simulated and observed events, it is necessary for design flood estimation that the distribution of the larger events is similar.
- A thorough investigation into the reliability of the observed streamflow data indicated that the data from many flow gauging stations were not suitable for verification studies. The importance of using a physically-based, conceptual model is highlighted by the unreliability of the flow data from a number of the gauging weirs. The direct calibration of a model to these data would result in erroneous simulations.
- The stages recorded at many of the flow gauges exceeded their rating tables during larger events, which are the focus of this study. The moderate extension of the rating tables performed in this study is believed to add to the uncertainty of the accuracy of the observed flow data.
- The annual maximum series (AMS) extracted from the primary flow data generated using the extended rating tables generally corresponded well with AMS supplied by Van Bladeren (2000), although in some cases the AMS from Van Bladeren exceeded the extracted values.
- No detailed information which could be used in a daily time step model was obtained on the transfers of water between catchments, which limited the number of catchments which could be used for verification of the simulations.
- The areas irrigated in the sub-catchments were derived from the Water Use Licencing, Registration and Revenue Collection database (WARMS). Without detailed information on irrigation management practices, two irrigation schedules were simulated and the results obtained indicate that the simulated volumes are not sensitive to different, but reasonable, irrigation scheduling. The

peak discharges obtained from the two irrigation schedules simulated were different as a result of the stochastic disaggregation of the daily rainfall.

- The development of irrigation in some sub-catchments impacted significantly on the streamflow simulated. This was evident in the results obtained from a number of catchments by good correspondence between accumulated observed and simulated runoff in the early part of the record when irrigation was not simulated, and in the latter part when irrigation was simulated.
- The use of a static land cover in the simulations, which does not reflect changes over time of land cover in a catchment, resulted in better simulations closer to the date of when the land cover information used in the simulations was captured.
- Despite the above issues, the distribution of simulated volumes generally compared well with the distribution of the observed data, over the range of catchment areas considered in this study (approximately 100 - 2000 km²). The distribution of the simulated peak discharges generally compared well with the observed distribution for the smaller catchments (< 150 km²), but were usually not as good for larger catchments. The results for the simulated design peak discharges reflect the results for the distribution of peak discharges, but are deemed to be reliable and consistent enough for use in practice.

Conclusions

A number of conclusions regarding the results from this study with respect to the application of the *ACRU* modelling system for use in design flood estimation can be made. These include:

- In order to realistically model the hydrology of the larger quaternary catchments at a daily time step, it is necessary to delineate the quaternary catchment into sub-catchments and, where appropriate, model each sub-catchment as a number of HRUs.
- The use of area weighted, and not modal, soils information is recommended.
- The accurate estimation of catchment rainfall is crucial to modelling the correct hydrological responses and the merging of raingauge and radar data is the best estimate of the true rainfall field.
- The use of historical rainfall from raingauges to estimate catchment rainfall can be improved by using relationships developed between the short record of merged rainfall fields and corresponding data from a selected raingauge.
- The SBM model holds much potential to generate detailed space and time stochastic rainfall values which can be used as input to a CSM system.
- The method of disaggregating daily rainfall developed in this project introduces a stochastic element into the *ACRU* model and the disaggregation procedures can be used by other hydrological studies where the disaggregation of daily rainfall is required.
- The use of the *ACRU* model for design flood estimation was shown to produce distributions of simulated daily volumes which generally reflected the observed distribution in catchments for a range of catchment areas.
- The distribution of the daily simulated peak discharges generally reflected the observed distribution in smaller catchments (< 150 km²). In larger catchments the good simulation of the distribution of streamflow volumes did not consistently translate into good distributions of daily peak discharge.

- The advantage of continuous simulation modelling to provide consistent and reliable estimates of design floods have been highlighted in this study. This approach avoids calibration against observed flow data which may be erroneous and is able to reflect current and future conditions in the catchments in the estimation of design floods and thus avoid any non-stationarity which may be present in the observed flow data.
- This study has emphasised the need to improve the estimation of historical rainfall, irrigation and land cover changes in a catchment in order to realistically simulate observed hydrological responses.
- The difficulty in translating a flood volume into a realistic hydrograph with the correct peak, particularly on larger catchments, is evident from the results obtained.

The results from this study have shown that the use of the *ACRU* model as a CSM can simulate the hydrological responses from an operational catchment, despite the challenges related to data and operations in the catchment. The output from the CSM has been shown to produce reasonable and consistent estimates of design floods, particularly in smaller catchments.

Recommendations for Future Research

In order to refine and improve the CSM for design flood estimation, the following recommendations are made:

- The method of adjusting historical raingauge data to estimate catchment rainfall should be applied for all catchments where radar data are available.
- The use of the merged rainfall fields created from the radar and raingauge data should be used as input to hydrological models where possible.
- In order to simulate historical flow data, the use of a time series of land cover information should be generated where possible and a database of changes to land cover should be captured and maintained for future use.
- An analysis of the hydrographs generated by the CSM should be made to further assess the performance of the model for design flood estimation.
- The translation of the runoff volume into a hydrograph and associated peak discharge requires further refinement. This may involve investigating the estimation of catchment lag and further investigation into the performance of flood routing algorithms for application in ungauged catchments.
- A more detailed study is required to assess the significance on the simulated performance of the stochastic disaggregation of daily rainfall.
- The SBM should be refined to be user friendly and easily calibrated against raingauge data and the SBM should be used as input to a CSM to generate long sequences of streamflow for design flood estimation.
- The methodology for the temporal disaggregation of daily rainfall requires further refinement. It is recommended that the methodology used be applied on discrete ranges of daily rainfall i.e. for smaller and larger events. It is further recommended sequencing of the disaggregated hourly rainfalls be refined in order to improve the simulation of the structure of the rainfall, as measured by the lag autocorrelations, number of events and event durations.

Knowledge Dissemination

Technology transfer has taken place in various forms during the course of the project. Conferences have been attended, papers have been published, guest lectures have been presented and MSc and MSc Eng dissertations have been completed. Below is the list of publications and dissertations produced, conferences attended and lectures presented by members of the project team.

Publications

- Chetty, K., Smithers, J. C. and Schulze, R. E., 2003. Towards a Continuous Simulation Modelling Approach for Design Flood Estimation in South Africa, 2nd IWRM Symposium. IAHS Red Book Series, Stellenbosch, South Africa, pp. 16.
- Chetty, K. T. and Smithers, J. C., 2005. Continuous Simulation Modelling for Design Flood Estimation in South Africa: Preliminary Investigations in the Thukela Catchment. *Physics and Chemistry of the Earth*, 30: 634-638.
- Frezghi, M. F. and Smithers, J. C., 2005. Merged rainfall fields for continuous simulation models, Proceedings of the Twelfth South African National Hydrological Symposium. SANCIAHS, Pretoria, RSA.
- Knoessen, D. M. and Smithers, J. C., 2005. The development and assessment of a daily rainfall disaggregation model for South Africa, Proceedings of the Twelfth South African National Hydrological Symposium. SANCIAHS, Pretoria, RSA.
- Schulze, R. E. and Ghile, Y. B., 2004. SCS-LHI: Background and User Manual on the Application of SCS-Based Techniques for the Estimation of Design Floods from Small Catchments in Regions with Limited Hydrological Information *ACRU Report No. 52*, School of Bioresources Engineering and Environmental Hydrology, University of KwaZulu-Natal, Pietermaritzburg, RSA.
- Smithers, J. C. and Chetty, K. T., 2005. Design flood estimation using a continuous simulation modelling approach, Proceedings of the Twelfth South African National Hydrological Symposium. SANCIAHS, Pretoria, RSA.
- Tewelde, M. H. and Smithers, J. C., 2006. Flood routing in ungauged catchments using Muskingum methods. *Water SA*, 32(3): 379-388.

Dissertations

- Frezghi, M. S., 2005. The development and assessment of a methodology to improve the estimation of the spatial distribution of rainfall. MSc Eng. Dissertation Thesis, School of Bioresources Engineering and Environmental Hydrology, University of KwaZulu-Natal, Pietermaritzburg, South Africa.
- Ghile, 2004. An adaptation of the SCS-ACRU hydrograph generating technique for application in Eritrea. MSc Dissertation Thesis, School of Bioresources Engineering and Environmental Hydrology, University of KwaZulu-Natal, Pietermaritzburg, South Africa, 167 pp.
- Knoesen, D. M., 2005. The Development And Assessment Of Techniques For Daily Rainfall Disaggregation In South Africa. MSc Dissertation Thesis, School of

Bioresources Engineering and Environmental Hydrology, University of KwaZulu-Natal, Pietermaritzburg, South Africa.

Tewolde, M. H., 2005. Flood routing in ungauged catchments using Muskingum methods. MSc Eng Dissertation Thesis, School of Bioresources Engineering and Environmental Hydrology, University of KwaZulu-Natal, Pietermaritzburg, South Africa.

Conferences attended and guest lectures

- IAHS 2nd Integrated Water Resources Management Symposium held in Stellenbosch, SA, 2003. (Ms Chetty)
- British Hydrological Society Symposium held in England, June 2005. (Prof Smithers)
- Guest Lecture, Centre for Ecology and Hydrology (CEH), Wallingford, UK, 2005 (Prof Smithers)
- Waternet Conference held in Namibia, November 2004 (Ms Chetty)
- Guest Lecture, Water Resources Development and Training Centre, University of Roorkee, India, January 2005 (Ms Chetty)
- SCS design flood estimation course, SAIAE Continuing Professional Development (CPD) Events: 2004, 2005 and 2006 (Prof Smithers, Prof Schulze and Ms Chetty)

Capacity and Competency Building

Capacity building has been practiced and successfully achieved throughout the duration of this project. The project employed Ms Chetty as the principal researcher on the project and has enabled her to complete her studies towards an MSc degree in Hydrology. The project also employed Mr Owen Wilson as a research assistant as part of a broader capacity building internship programme within the School of Bioresources Engineering and Environmental Hydrology at the University of KwaZulu-Natal. Mr Wilson subsequently successfully completed his Honours degree in Hydrology. Both Mr Wilson and Ms Chetty are from previously disadvantaged communities.

Four students, Mr Mehari Frezghi (MSc Eng), Mr Darren Knoesen (MSc) and Mr Mesfin Tewolde (MSc Eng) and Mr Yonas Ghile (MSc), have completed their postgraduate degrees at the University of KwaZulu-Natal while working on components of the project.

Both Ms Chetty and Professor Smithers, the project leader, teach Hydrology and Bioresources Engineering modules at undergraduate and postgraduate levels and the students employed on the project tutored modules at undergraduate level. Currently over 70% of these classes comprise of students from previously disadvantaged backgrounds.

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1. INTRODUCTION

Estimates of design floods are required by engineers, hydrologists and agriculturalists for the design of hydraulic structures such as dams, bridges or culverts (Smithers and Schulze, 2003). Under or over-design of even small hydraulic structures can result in a considerable waste of resources. The choice of an acceptable and cost-effective engineering solution is dependent upon having reliable estimates of the frequency of floods, both in terms of peak flows and volumes of water.

Recent floods such as those in February 2000 which occurred in the North-Eastern part of South Africa, Zimbabwe and Mozambique and flooding in the Western Cape in 2005 highlight the need to assess the risks associated with floods in South Africa. Design flood estimation, where the magnitude of a flood is associated with a level of risk (e.g. return period) is necessary in the design of hydraulic structures.

Reliable estimates of flood frequency in terms of peak flows and volumes remain a challenge in hydrology (Cameron *et al.*, 1999). Cordery and Pilgrim (2000) express the opinion that the demands for improved estimates of floods have not been met with any increased understanding of the fundamental hydrological processes. The urgency for new approaches to design flood estimation in South Africa has been highlighted by Alexander (2002) and Smithers and Schulze (2003).

Standard techniques for flood estimation have been developed for most countries and procedures for design flood estimation may be broadly categorised as methods based on the analysis of observed floods and rainfall based methods. The situation which faces design engineers and hydrologists most frequently is when no, or inadequate, observed streamflow data are available at the site of interest and then either regional approaches or rainfall-runoff models have to be used for design flood estimation. Reviews of approaches to design flood estimation are contained in Cordery and Pilgrim (2000), Smithers and Schulze (2001) and Görgens (2002).

Rainfall data are far more abundant and have longer records than streamflow in South Africa. Hence, rainfall event-based approaches such as the Unit-Hydrograph, Rational Method and Soil Conservation Service (SCS) methods, which are generally simple to apply and which lump complex, heterogeneous catchment processes into a single process or index, are currently widely used for design flood estimation in South Africa. A major limitation of event-based methods is the assumption that the exceedance probability of the simulated streamflow is the same as that of the input design rainfall. Numerous studies have shown that this is generally not the case and that the antecedent soil moisture conditions prior to an event are significant in determining the runoff response (Schulze and Arnold, 1979; Schulze, 1982; Schmidt and Schulze, 1984; Dunsmore *et al.*, 1986; Pilgrim and Cordery, 1993). Another major limitation of event-based methods is the inability of most of the methods to account for antecedent soil moisture conditions prior to flood events, which may result in unrealistic estimates of runoff. A single estimate of design rainfall is used as input to event-based models to estimate the design flood over a catchment. The single event rainfall input may be a limitation of the approach as recent experiences in South Africa suggest that multi-catchment storms have been responsible for floods which have caused large scale destruction and loss of life.

The limitations of event-based approaches to design flood estimation can be overcome by adopting a continuous simulation approach to rainfall-runoff modelling. Continuous Simulation Models (CSM) strive to represent the major processes

responsible for converting the input catchment rainfall into streamflow (Rahman *et al.*, 1998). Hydrographs are generated for long periods of time using the input historical rainfall series, potential evaporation and other climatological as well as catchment attributes (e.g. soils, land cover and topography). An important characteristic of these models is the use of a continuous water budget model for the catchment so that antecedent conditions prior to each rainfall event is simulated (Rahman *et al.*, 1998). A relatively sophisticated hydrological model capable of simulating the entire hydrological cycle is required by this approach to design flood estimation. This implies a more complex model calibration effort and generally requires extensive data and processing (ASCE, 1997).

In practice the entire design hydrograph, and not only the peak discharge, is frequently required when designing, for example, flood reducing detention ponds and emergency spillways for dams. The output from a continuous simulation approach generally includes the entire hydrograph which can be analysed directly, and hence the computed exceedance probability of a flood is not dependent on the exceedance probability of the input rainfall. Current or predicted patterns of rainfall and land use in the catchment may be also modelled explicitly and thus influence the estimated design floods. Runoff from rainfall varying over multiple catchments can be simulated and analysed. Antecedent soil moisture conditions are modelled explicitly and hence realistic runoff responses to rainfall events can be obtained. An additional advantage of the continuous simulation modelling approach is that the flood attenuating effects of, for example, river reaches and dams can be directly accounted for by the model. These and other advantages of continuous simulation modelling for design flood estimation are widely reported in the international literature (e.g. Boughton and Hill, 1997; Rahman *et al.*, 1998; Cameron *et al.*, 1999; Reed, 1999) and continuous simulation modelling has been used for design flood estimation in a number of international studies (Calver and Lamb, 1995; Boughton and Hill, 1997; Calver *et al.*, 1999; Cameron *et al.*, 1999; Lamb, 1999; Steel *et al.*, 1999; Calver *et al.*, 2000; Calver *et al.*, 2001; Calver *et al.*, 2004) and has also been demonstrated in South Africa (Smithers *et al.*, 1997; Smithers *et al.*, 2001). According to Rahman *et al.* (1998) some of the disadvantages of CSM include the loss of "sharp" events if the modelling time scale is too large, the extensive data requirements which result in significant time and effort to obtain and prepare the input data, and the expertise required to determine parameter values such that historical hydrographs are adequately simulated. Despite these disadvantages, CSM may prove to be the most powerful means of estimating flood frequency from rainfall (Rahman *et al.*, 1998) and CSM for design flood estimation is receiving increasing interest and use in the USA (ASCE, 1997). Calver *et al.* (2000) express the opinion that CSM may form the basis for the next generation of flood frequency estimation in the UK.

The objectives of the project, as stated in the contract, were to develop a regionalised, continuous simulation modelling system for the estimation of the frequency of floods in South Africa. The computed exceedance probabilities of floods would not be inferred from rainfall, but would reflect the joint associations and complex interrelationships between rainfall, antecedent soil moisture conditions and runoff. It was envisaged that the system would also have the ability to account for current or future land covers and management practices and the generation of long time series of runoff, using either historical or stochastic rainfall series, will decrease the uncertainties in the estimated design floods.

The primary and secondary research products envisaged from this project were the following:

- A methodology for the estimation of the frequency of floods in South Africa which accounts for the joint association between rainfall, antecedent soil moisture conditions and runoff and reflects current or probable future land covers and management practices.
- A methodology based on continuous simulation modelling for the estimation of the frequency and spatial distribution of droughts in South Africa.

However, at the inaugural meeting (18 March 2003) the Steering Committee recommended that, given the scope of the project and resources available, the secondary product (droughts) should not be considered in this project and hence no further development of the system for droughts was pursued.

In this document the following aspects of the project are reported:

- (i) Chapter 2 contains a review of approaches to design flood estimation with particular reference to the use and application of continuous simulation modelling.
- (ii) The development of different components of the continuous modelling is outlined in Chapter 3.
 - Results from investigations into the scale of application and levels of soils and land cover information necessary to apply continuous simulation modelling for design flood estimation are reported in Section 3.2.
 - The development and evaluation of a method to disaggregate daily rainfall into hourly totals in South Africa is reported in Section 3.3.
 - Section 3.4 contains an assessment of merged raingauge and radar data to estimate the spatial distribution of rainfall and the use of improved estimates of catchment rainfall for modelling. In addition, an initial assessment of the stochastic, fine resolution, space-time String-of-Beads model to simulate catchment rainfall has been performed and the results are also reported in Section 3.5.
 - The development and assessment of techniques for flood routing in ungauged catchments are reported in Section 3.6.
 - The development and application of climate-based adjustments to Curve Numbers used in the SCS model for antecedent soil moisture conditions is reported in Section 3.7
- (iii) The application of the above developments for design flood estimation in the Thukela Catchment is reported in Chapter 4.
- (iv) Chapter 5 contains discussion on the document, conclusions and recommendations for further research.
- (v) A list of references used in the document is contained in Chapter 6.

2. DESIGN FLOOD ESTIMATION

A number of different approaches to design flood estimation are commonly used both in South Africa and internationally. The selection of method to use is based on consideration of the following (Pilgrim and Cordery, 1993):

- the theoretical basis of the method, the factors taken into account and the relative accuracies of the methods,
- whether a deterministic or probabilistic estimate is required, and the suitability of the method and parameter estimates to the present application,
- whether the method can be calibrated from local data or, if it is a regional method, was the method developed using data recorded in the region,
- the importance of the application and the effect of inaccuracies,
- whether a peak flow or complete hydrograph is required,
- the time available for estimating the flood, and
- the expertise available for the more complex methods.

This chapter will provide a brief review of methods for design flood estimation and will introduce the concept of Continuous Simulation Modelling (CSM) for design flood estimation and developments both in South Africa and internationally will be reviewed. Much of the material and structure of this chapter have been extracted from Smithers and Schulze (2003) and Chetty (2003).

2.1. Methods for Design Flood Estimation

The categorisation of approaches to design flood estimation has been attempted by numerous authors. According to HRU (1972), design floods may be estimated using either a statistical approach, which is an ordering and transposition of past experience, or a deterministic approach, in which rainfall is translated into a flood.

SANRA (1986) identified empirical, statistical, as well as the Rational, SCS, run-hydrograph and synthetic unit hydrograph approaches as appropriate and promising methods for estimating design floods in South Africa. Alexander (1990; 2001) classified methods of design flood estimation in South Africa as direct statistical analysis, regional statistical analysis, deterministic, and empirical methods. Pegram (1994) identified deterministic, empirical and statistical methods and presented a decision tree for the selection of method for design flood estimation in South Africa. In addition to these approaches, the ASCE (1997) summarise the use of simplified methods such as formulae, regression equations and envelope curves, and also include rainfall-runoff analysis for a period of record where a historical sequence of rainfall is input to the model to generate the variable of interest, which can then be subjected to frequency analyses, i.e. continuous simulation modelling.

The Flood Estimation Handbook (FEH) for the UK provides two main approaches to flood frequency estimation (Reed, 1999). The first is an index flood approach which utilises growth curves and is the first choice when there is a long record of gauged flow at or close to the site of interest, and which may be used for catchments with areas larger than 0.5 km². The second approach is the Flood Studies Report (FSR) rainfall-runoff method which may be used for catchments with areas up to 1000 km².

Cordery and Pilgrim (2000) differentiate between statistical analyses, applied either at a single location or across a region, and either deterministically or probabilistically based rainfall-runoff modelling for flood estimation. Beven (2000) distinguishes

between statistical estimation based on samples of observed floods at a site, regionalisation methods for catchments with no data, and methods based on rainfall-runoff modelling. The methods used for design flood estimation are categorised and summarised in Figure 2.1.

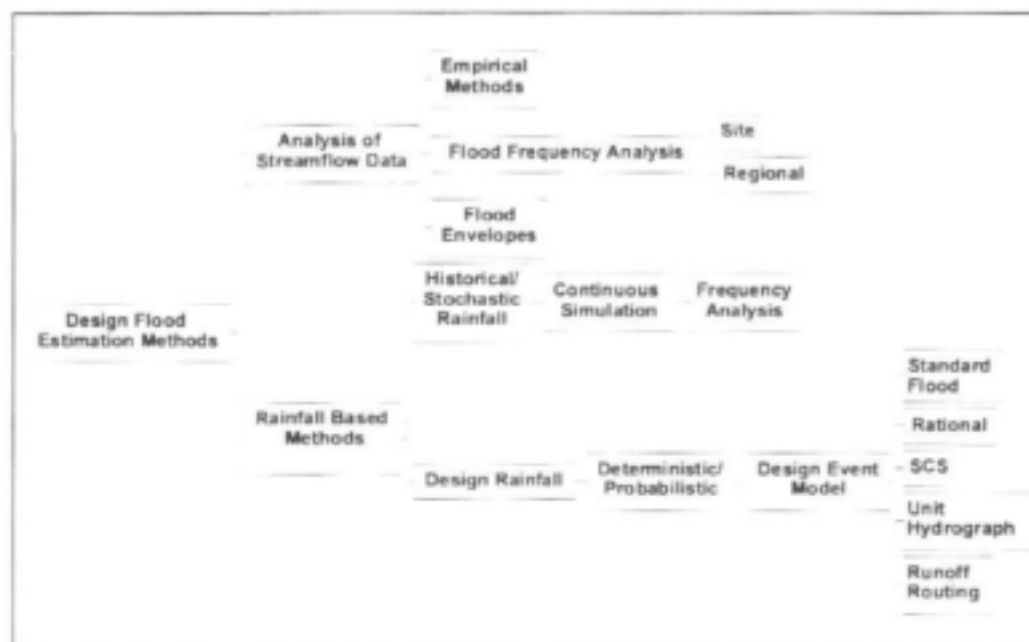


Figure 2.1 Methods for estimating design floods (after Smithers and Schulze, 2001)

The two main approaches to design flood estimation are methods based on the analysis of the streamflow data and rainfall-runoff based methods. Methods based on the analysis of streamflow data (flood data) are used when there are adequate streamflow data available and these methods can be divided into empirical formulae (e.g. Roberts, 1963; Roberts, 1965), maximum flood envelopes (e.g. Kovacs, 1988) and flood frequency analysis which may be performed using either an at-site or regional approach. When the streamflow data are inadequate, or no streamflow data are available at the site of interest, then either event-based or continuous rainfall-runoff modelling methods are used for design flood estimation.

2.2. Analysis of Streamflow Data

Flood data may be used to estimate design floods either at a site or within a region. The methods are inherently probabilistic and are therefore suitable for estimating floods for design purposes (Cordery and Pilgrim, 2000). As shown in Figure 2.1, the methods can be classified as empirical, flood envelopes and flood frequency analyses.

2.2.1. Empirical Methods

Empirical methods use empirical formulae which generally relate peak discharge to catchment size and other physiographical and climatic catchment characteristics. According to Rahman *et al.* (1998), these methods are a "black box" model type i.e.

they do not incorporate any hydrologic knowledge in the system, and are simply a statistical means of converting a known rainfall input into a design flood output.

The Creager method and the Francou-Rodier Methods are classified by Alexander (1990) as empirical methods. Roberts (1963; 1965), cited by Alexander (1990), developed a method to estimate design peak discharges in South Africa as a function of catchment area, a catchment coefficient and a coefficient derived from the Hazen distribution. Pitman and Midgley (1967) identified 7 homogeneous flood producing regions in South Africa and developed a co-axial diagram with four variables (return period, locality, catchment area and peak discharge) to estimate design floods in South Africa. Herbst (1968), cited by Alexander (1990), developed the relationship further and also included mean annual precipitation (MAP) and coefficient of variation of the floods as variables. USGS quantile regression methods and the Probabilistic Rational Method used in Australia are also examples of empirical methods (Rahman *et al.*, 1998).

Empirical methods are generally used for maximum flood estimation and estimates therefore tend to be conservative (Alexander, 1990). Coefficients can be derived to directly link the flood and rainfall exceedance probabilities and the empirical methods overcome this limitation of many of the design event models. However, this limits their application to within the range of conditions where they were calibrated (Rahman *et al.*, 1998). If these methods are not calibrated from the catchment of interest, then their extrapolation can be hazardous and should not be used (Cordery and Pilgrim, 2000). According to the National Transport Commission (TNC, 1981), empirical and experienced based methods should only be used for checking other methods of design flood estimation.

2.2.2. Maximum Envelopes

In the maximum envelope approach, the largest observed discharges are usually plotted against catchment area, both on logarithmic axes. An envelope is sketched to include all the data points. Approximate estimates are possible provided that data from catchments similar to the one of interest are included in the analysis (Cordery and Pilgrim, 2000). Maximum peak discharges can be estimated at ungauged sites using envelope curves (ASCE, 1997). The envelope tends to increase as the record length increases and larger floods are observed.

HRU (1972) provided a set of regionalised maximum observed flood peak envelopes for South Africa. Kovacs (1988) developed comprehensive regional maximum flood envelopes for South Africa based on the Francou-Rodier approach. This approach is believed to be reliable in medium sized catchments (Alexander, 1990).

2.2.3. Flood Frequency Analysis

Flood frequency analysis remains a subject of great importance and interest (Bobee and Rasmussen, 1995). This approach to design flood estimation can be split into at-site and regional flood frequency analyses, and both approaches are discussed in the following sections.

2.2.3.1. At-site Analysis

The procedures for direct frequency analysis of observed peak discharge requires the selection and fitting of an appropriate theoretical probability distribution to either the annual maximum series or partial duration series which have been extracted from the

observed data. These procedures are referenced in standard hydrology texts (e.g. Chow *et al.*, 1988; Stedinger *et al.*, 1993). Goodness-of-fit tests are performed to assess which probability distribution best fits the data and the selected distribution is used to provide appropriate estimates of design quantiles (Smithers, 1996).

As shown by Schulze (1989) and Smithers and Schulze (2000b), the question of selecting an appropriate distribution has received considerable attention in the literature, with diverging opinions expressed by various authors. Schulze (1989) questions whether a suitable probability distribution can be selected, given that the best distribution varies with, *inter alia*, the season, storm type and duration and regional differences.

Smithers and Schulze (2000b) summarise approaches available for estimating the parameters of a selected distribution as Method of Moments (MM), Maximum Likelihood Procedure (MLP), Probability Weighted Moments (PWM), L-Moments (LM), Bayesian Inference and non-parametric methods. The use of L-moments to fit distributions has received extensive coverage in the literature (e.g. Wallis, 1989; Hosking, 1990; Pearson *et al.*, 1991; e.g. Gingras and Adamowski, 1992; Guttman, 1992; Pilon and Adamowski, 1992; Guttman, 1993; Guttman *et al.*, 1993; Lin and Vogel, 1993; Vogel and Fennessy, 1993; Vogel *et al.*, 1993a; Vogel *et al.*, 1993b; Wallis, 1993; Gingras and Adamowski, 1994; Zrinji and Burn, 1994; Hosking, 1995; Hosking and Wallis, 1995; Karim and Chowdhury, 1995; Hosking and Wallis, 1997). L-moments are reported to have less bias when compared to other techniques.

Bobee and Rasmussen (1995) describe the use of L-moments for distribution fitting as an "eye-catching" development for flood frequency analysis while Cordery and Pilgrim (2000) "welcome" the developments of L-moments. However, Bobee and Rasmussen (1995) caution that L-moments may be too robust and outliers may be given too little significance, while Cordery and Pilgrim (2000) emphasise that the use of L-moments does not entirely overcome the fundamental problem of selecting an appropriate distribution for a sample from a population with an unknown distribution.

Schulze (1989) highlights the problem of short data sets and extrapolation beyond the record length. He also illustrates typical measurement errors as well as inconsistency, non-homogeneity and non-stationarity of data, all of which violate the assumptions made when fitting a distribution to the data.

Beven (2000) identifies the following limitations of a direct statistical approach:

- The correct distribution of the flood peaks is unknown and different probability distributions may give acceptable fits to the available data, but result in significantly different estimates of design floods when extrapolated.
- The records of gauged runoff are generally short and the calibration of the gauging structures may not be very robust. Hence the sample only represents a small distribution of the floods at the site and the fitted distribution may be further biased by gauging errors.
- The frequency of flood-producing rainfalls and land use characteristics may have changed during the period of historical measurement.
- The fitted distribution does not explicitly take into account any changes in the runoff generation processes for higher magnitude events.

2.2.3.2. Regional Analysis

Given that the data at a site of interest will seldom be sufficient, or available, for frequency analysis, it is necessary to use data from similar and nearby locations (Stedinger *et al.*, 1993). This approach is known as regional frequency analysis and utilises data from several sites to estimate the frequency distribution of observed data at each site (Hosking and Wallis, 1987; Hosking and Wallis, 1997). Regional frequency analysis assumes that the standardised variate has the same distribution at every site in the selected region and that data from a region can thus be combined to produce a single regional flood, or rainfall, frequency curve that is applicable anywhere in the region with appropriate site-specific scaling (Cunnane, 1989; Gabriele and Arnell, 1991; Hosking and Wallis, 1997). Regionalisation enables a frequency analysis of short records of annual floods to be performed by assisting with the identification of the shape of the parent distribution and leaving the measure of scale to be estimated from the at-site data (Bobee and Rasmussen, 1995).

In the context of flood frequency analysis, regionalisation refers to the identification of homogeneous flood response regions and the selection of an appropriate frequency distribution for the selected regions (Kachroo *et al.*, 2000). Within a homogeneous region, historical data can be pooled to obtain efficient estimates of the parameters of the distribution and hence robust quantile estimates (Kachroo *et al.*, 2000) with smaller standard errors (Mkhandi *et al.*, 2000). Thus, the concept of regional analysis is to supplement the time limited sampling record by the incorporation of spatial randomness using data from different sites in a region (Schaefer, 1990; Nandakumar, 1995).

Regional approaches can also be used to estimate events where no information exists (ungauged) at a site (Pilon and Adamowski, 1992). However, care must be exercised to ensure that such an approach is not applied outside of the region where the method was developed, nor outside of the range of observations used to develop the method (Cordery and Pilgrim, 2000).

In nearly all practical situations a regional method will be more efficient than the application of an at-site analysis (Potter, 1987). This view is also shared by both Lettenmaier (1985; cited by Cunnane, 1989) who expressed the opinion that "regionalisation is the most viable way of improving flood quantile estimation" and by Hosking and Wallis (1997) who, after a review of literature up to 1996, advocate the use of regional frequency analysis based on the belief that a "well conducted regional frequency analysis will yield quantile estimates accurate enough to be useful in many realistic applications". This opinion is also expressed by Cordery and Pilgrim (2000), who conclude that regional approaches are "the only sure basis for improved flood prediction". According to Alexander (1990), regional statistical analyses provide a basis for improving the estimates of the parameters of the distribution at both gauged sites with short records and at ungauged sites. The advantages of regionalisation are thus accepted by numerous respected researchers.

The index flood-based procedure developed by Hosking and Wallis (1993; 1997) and which utilises L-moments appears to be a robust procedure and has been applied in a number of studies. For example, the methodology has been successfully applied by Smithers and Schulze (2000a; 2000b) to estimate both short and long duration design rainfalls in South Africa. A cluster analysis of site characteristics is used to identify potential homogeneous regions, which allows for independent testing of the at-site data for homogeneity. Methods based on L-moments are used for frequency

estimation, screening for discordant data and testing clusters for homogeneity (Hosking and Wallis, 1993; Hosking and Wallis, 1997).

Much research in recent years has focussed on the identification of homogeneous regions, as geographical proximity does not imply hydrological similarity (Bobee and Rasmussen, 1995). Kachroo *et al.* (2000) reviewed recent literature and concluded that no objective methods of regionalisation are universally accepted. A significant development in the identification of homogeneous regions is the region of influence approach developed by Burn (1990a; 1990b) and Zrinji and Burn (1990a; 1990b) and which has been adopted by the FEH (Reed, 1999).

The recommended distribution for flood frequency analyses in the USA is the log-Pearson Type 3 (LP3), fitted using the at-site mean and standard deviation and a regionalised estimate of the coefficient of skewness (Stedinger *et al.*, 1993). Details are contained in USWRC (1976) and updated in the subsequent Bulletin 17B publication (IACWD, 1982) which includes procedures for dealing with outliers and conditional probability adjustment. Potter and Lettenmeier (1990) showed that an index flood approach using a GEV distribution performed better than the procedures contained in Bulletin 17B.

According to Alexander (1990) no comprehensive studies of regional statistical analysis methods have been made in South Africa since the early 1970s. He outlines a generalised procedure for regional statistical analyses. According to Alexander (1990), the distribution of gauging stations in South Africa is too sparse to pre-determine hydrologically homogeneous regions. He recommends an alternative method of grouping stations which consists of plotting scaled growth curves and rejecting stations which have growth curves inconsistent with the remaining stations.

McPherson (1984) investigated methods to estimate the mean annual and two year return period floods in South Africa. The catchment parameter method developed showed promising initial results, but has not been developed further.

A tentative regionalisation based on the regions identified by Kovacs (1988) was performed by Van Bladeren (1993) for the KwaZulu-Natal and former Transkei regions. He noted that further regionalisation was necessary and that a strong relationship existed between the mean annual flood and catchment area.

Mkhandi *et al.* (2000) used the L-moment based procedures developed by Hosking and Wallis (1993) to identify both discordant gauging stations and homogeneous flood producing regions in Southern Africa. Thirteen homogeneous regions were delineated utilising drainage regions in South Africa and the Pearson Type 3 distribution fitted by PWM was found to be the most appropriate distribution to use in 12 of the regions. In the western coastal region of South Africa the LP3 distribution fitted by MM was found to be the most appropriate distribution.

A initial regionalisation using the annual maximum series of peak discharges for KwaZulu-Natal in South Africa was derived by Kjeldsen *et al.* (2002). The index flood method, as proposed by Hosking and Wallis (1993; 1997) was utilised in the study. Two homogeneous regions were identified and suitable regional frequency distributions were sought. In order to estimate a design flood at an ungauged site, it is necessary to re-scale the regional growth curve by an estimate of the index flood at the site. Kjeldsen *et al.* (2001) developed relationships to estimate the index flood as a function of the MAP and catchment area.

The run-hydrograph technique as detailed in Hiemstra *et al.* (1976), Hiemstra and Francis (1979) and Hiemstra (1981), is based on a regional analysis of historical data but was recommended, soon after its development, only to be used to check the results from other methods (SANRA, 1986). Although no further evaluation of the method has been documented since the report by SANRA (1986), Alexander (1990) does not recommend the run-hydrograph procedure for general use in South Africa, while Alexander (2001) concedes that the run-hydrograph method has advantages compared to the unit hydrograph method and concludes that the run-hydrograph method requires further development. The run-hydrograph method is endorsed for use in South Africa by Pegram (1994) and is currently being revised and updated for application in South Africa by van Rensburg (2005).

2.3. Rainfall Based Methods

According to Beven (2000), rainfall-runoff processes are modelled as it is not possible to measure all hydrological processes and hence modelling is a means of extrapolating both in space and time from those measurements which are made to provide some prediction for decision making. The situation which faces design engineers and hydrologists most frequently is when no, or inadequate, streamflow data are available at the site of interest. As indicated in Figure 2.1, the choices available in such a situation are between event and continuous rainfall based methods. Both deterministic and probabilistic models are used. The advantages of rainfall-runoff models may be summarised as follows (Schulze, 1989; Rahman *et al.*, 1998):

- Generally longer rainfall records at more sites, and with better quality, are available for analysis compared to streamflow records.
- Measurement errors, inconsistencies in the data and non-homogeneous streamflows make the data unsatisfactory for direct frequency analysis.
- Similarly, non-stationary streamflow records as a result of changing catchment conditions can render the streamflow record unsatisfactory for direct frequency analysis.
- Areal extrapolation of rainfall records can be achieved more easily than runoff records.
- Physical features of a catchment can be incorporated into a rainfall-runoff model.
- The historical, current or expected future conditions of land use within a catchment can be modelled.

2.3.1. Design Event Models

Design event based models assume that, for representative inputs and model parameters, the frequency of the estimated flood is equal to the frequency of the input rainfall. This assumption is likely to introduce significant bias in the frequency of flood estimates and the validity of this assumption is crucial to the accuracy of this approach (Rahman *et al.*, 1998). Much uncertainty is present in inputs such as storm duration, the spatial and temporal distribution of the design storm and model parameters (Rahman *et al.*, 1998). Design event based approaches consider the probabilistic nature of rainfall, but ignore the probabilistic behaviour of other inputs and parameters. Four general approaches are suggested by Pilgrim and Cordery (1993) to maintain the required probability for the selected flood, with the last two options listed below showing the greatest practical value:

- frequency analysis of synthetic streamflow generated by a continuous rainfall-runoff from long records of rainfall,
- joint probability analysis of variables contributing to the flood discharge,
- use of median values for model parameters, and
- values derived by comparison of floods and rain of the same probability.

Three approaches have been adopted in Australia in an attempt to estimate a flood with the required return period (Pilgrim, 1987; Pilgrim and Cordery, 1993):

- The use of median values of input, other than rainfall, to models:
- The use of "worst" design parameters (e.g. high runoff coefficients and low losses) result in flood estimates with exceedance probabilities lower than that of the input rainfall.
- The derivation of relationships that link directly the rainfall and runoff for the same exceedance level:
- An example of this approach being the probabilistic approach to the Rational Method.
- The use of joint probabilities to the variables which contribute to the flood:
- Although superior to the above two approaches, uncertainties increase for larger return periods.

For small to medium sized catchments in Australia, Pilgrim (1987) recommends the probabilistic Rational Method with regionalised coefficients, regional flood frequency methods, design hydrograph methods which include unit hydrograph and loss function methods, synthetic unit hydrographs and runoff routing and the USDA's SCS technique as appropriate methods for design flood estimation. The Rational, SCS, Gradex, Unit Hydrograph and runoff-routing are listed by Cordery and Pilgrim (2000) as commonly used design rainfall event methods for flood estimation. These are briefly discussed in the following sections.

The widespread use of design event models is related to their lumping of complex, heterogeneous catchment processes into a single process, their ability to handle individual events, and simple model application (Houghton-Carr, 1999). The event based approach greatly simplifies the estimation of catchment conditions prior to the occurrence of an extreme event, even when rainfall-runoff modelling is performed to estimate a flood hydrograph (Cameron *et al.*, 1999).

2.3.1.1. Rational Method

The Rational Method is widely used throughout the world for both small rural and urban catchments (Pilgrim and Cordery, 1993; Alexander, 2001). The Rational Method is viewed as an approximate simplified technique for design flood estimation in the USA which requires little effort to apply (ASCE, 1997). The method is an approximate deterministic method and a major weakness is the judgement required to determine the appropriate runoff coefficient and the variability of the coefficients between different hydrological regimes (Pilgrim and Cordery, 1993). The Rational Method computes only flood peaks and is sensitive to the input design rainfall intensity, the selection of the runoff coefficient, the experience of the user and should not be used for catchments $> 15 \text{ km}^2$. The experience of the user and the selection of appropriate runoff coefficients are essential for the application of the Rational Method, which can give realistic results when used circumspectly (SANRA, 1986). In

addition, Cordery and Pilgrim (2000) identify the practical difficulties of estimating the catchment response time because regional differences in the time of concentration cannot be easily explained by measured catchment characteristics. The assumed uniform rainfall intensity and the exclusion of temporary storage limits the application of the deterministic Rational Method to urban and small rural catchments (Cordery and Pilgrim, 2000). Hence, Pilgrim and Cordery (1993) and Cordery and Pilgrim (2000) recommend a probabilistic approach to determine the runoff coefficient for the Rational Method.

The probabilistic Rational Method has been developed for Australia with the runoff coefficient for different return periods either mapped or related by regression to catchment based physical variables. Studies in Australia have shown the superior performance of the probabilistic Rational Method, which is suitable for catchments of up to 250 km², compared to the very poor performance of the deterministic approach (Pilgrim and Cordery, 1993). Contrary to the deterministic approach, the probabilistic runoff coefficients did not show much variation with catchment characteristics (Pilgrim and Cordery, 1993). According to Pilgrim (1987), the use of the probabilistic interpretation of the Rational Method is acceptable for estimating design events, but the method is not suitable for estimating the flood peak for a particular rainfall event.

The HRU (1972) outlines a deterministic Rational Method approach to design flood estimation in South Africa which is suitable for application in catchments with areas of up to 15 km² and the Rational Method is also recommended by SANRA (1986) and Alexander (1990; 2001). The runoff coefficient may be estimated as a function of MAP, catchment land cover, permeability and steepness, vegetation cover and return period. The return period adjustment factor decreases the runoff coefficient for events with return periods < 50 years. Differing values for the return period adjustment factor in South Africa are presented by SANRA (1986) and Alexander (2001). Alexander (1990) advocates the calibration of the Rational Method with local data, where it is available.

Alexander (2002) developed a "Standard Design Flood" method, which is in effect a calibrated Rational Method. Raingauges were assigned to 29 representative catchments in South Africa which have observed flow data and the Rational runoff coefficient ("C" factor) was calibrated until the design flood estimated using design rainfall values equalled that computed directly from the gauged flow data. Some subjective adjustment was performed to the calibrated runoff coefficients to "produce a more conservative estimate". The 29 catchments were grouped into 8 larger regions and verifications were performed at 84 sites where, on average, the standard design flood exceeded the at-site values by 60%. According to Grgens (2002) the "Standard Design Flood" method is a conservative approach and would not be suitable, for example, in the design of dam spillways.

The accurate estimation of the of the runoff coefficient is crucial to the use of the Rational Method (van Rensburg, 2005). Pegram (2003) showed that Rational Method could be extended to much larger catchments than commonly accepted. Parak and Pegram (2005) showed that the method is applicable to any sized catchment, provided that a probabilistic approach to the estimation of the runoff coefficient in the Rational Method is utilised.

2.3.1.2.SCS Method

Estimates of design floods are frequently required for small catchments where the data are generally inadequate data (Pilgrim, 1987; Marshall and Bayliss, 1994). According to Schulze *et al.* (1992) there is a frequent need for hydrological information for use in the planning, design and management of water resources systems on small catchments, i.e. with areas $< 30 \text{ km}^2$. This has led to the wide use of the SCS method for design flood estimation which has, in the USA, replaced the Rational Method (Pilgrim and Cordery, 1993). This is attributed by Pilgrim and Cordery (1993) to the wider apparent database and the manner in which the physical catchment characteristics are incorporated in the method. Inconsistencies in the application of the method are the result of the choice of procedures for estimating the time of concentration and in choosing a relevant curve number (CN). Pilgrim and Cordery (1993) summarise the following with regard to the SCS method:

- The SCS model performed poorly in simulating actual peak discharges from runoff plots in the USA.
- The assumed antecedent moisture conditions had a major effect on the results.
- The model performed better on catchments with sparse vegetation than those with dense vegetation.
- The SCS method was applied in a probabilistic manner in Australia and the derived CN showed little agreement with those estimated by conventional means. The derived CN was affected both by the method used to estimate the catchment lag time and on the return period.

The above results led Pilgrim and Cordery (1993) to question the accuracy and validity of the SCS method and suggest that the results from the SCS method should be checked against observed flood data in the region in which it is applied. Cordery and Pilgrim (2000) express the opinion that the SCS method is vaguely intuitive and cannot be expected to provide reliable design estimates.

Haan and Schulze (1987) treated the input variables in the SCS equation as random variables in order to correctly transform the rainfall with a given exceedance probability into runoff with the same exceedance probability. They found that the traditional SCS method of accounting for antecedent moisture conditions resulted in reasonable estimates of runoff.

The SCS method adapted for South Africa by Schmidt and Schulze (1987) utilised the developments and verifications by Schulze and Arnold (1979), Schulze (1982), Schmidt and Schulze (1984) and Dunsmore *et al.* (1986). These adaptations were computerised by Schulze *et al.* (1992) and the SCS-SA method is now widely used for the estimation of design floods from small catchments in South Africa. The SCS method is not as sensitive as the Rational Method to user inputs. It can compute the entire hydrograph and is recommended for both urban and rural catchments with areas $< 10 \text{ km}^2$ (Campbell *et al.*, 1986; SANRA, 1986). A further statistical analysis of the results presented by Campbell *et al.* (1986) was performed by Schulze *et al.* (1986), who excluded rainfall events less than 20 mm, and concluded that the SCS-based models, particularly the South African adaptations, performed well enough to be recommended for design on a considerable range of land use and catchment size categories.

2.3.1.3. Unit Hydrograph Method

The unit hydrograph approach to design flood estimation is detailed in most hydrology texts (e.g. Chow *et al.*, 1988; Maidment, 1993). The method assumes a characteristic linear response from a catchment and hence may not be accurate for estimating large floods in catchments which have a non-linear hydrological response. However, careful use can provide good flood estimates. A limitation of a unit hydrograph approach is the assumption of spatial uniformity of rainfall (Chow *et al.*, 1988; Maidment, 1993). An advantage of the method is the estimation of the entire hydrograph, which is important where storage within a catchment is present.

In the UK the FEH re-states the FSR rainfall-runoff method for design flood estimation with new estimates of model parameters (Houghton-Carr, 1999). The FSR model is a deterministic 3 parameter model of catchment response and consists of a unit hydrograph and a loss model (Houghton-Carr, 1999). The parameters relate to the catchment response time, the proportion of rainfall which contributes directly to flow in the river and the quantity of baseflow in the river prior to the event. The parameters for the model may be derived from observed rainfall and runoff records if these are available or, at ungauged sites, either from physical and climatic descriptors of the catchment or by the transfer of information from gauged "donor" catchments. The point design rainfall for the required return period flood is converted to an areal rainfall using an areal reduction factor and disaggregated temporally to form a hyetograph. The estimated runoff is converted into a hydrograph using the catchment unit hydrograph and baseflow is added to complete the design hydrograph (Houghton-Carr, 1999).

The FSR unit hydrograph and loss model is widely used for three reasons (Houghton-Carr, 1999):

- the model is relatively well understood,
- the model can generally be easily derived for any site, and
- the simple structure of the model allows the incorporation of local data.

Houghton-Carr (1999) identifies the most general weakness of the FSR model to be the assumption that a unique combination of the four specific inputs will estimate a design flood with the required return period. The performance of the original FSR model has been shown to vary regionally, which is attributed to the approach adopted for model calibration, with the calibrations performed at a national scale (Houghton-Carr, 1999). No evaluations of the current versions of the model have been made (Houghton-Carr, 1999).

For catchment areas ranging from 15 - 5000 km², the HRU (1972) describes a unit hydrograph technique for application in South Africa, which was updated by Bauer and Midgley (1974). Data from only 92 gauges with catchment areas ranging from 21 to 22163 km² were used in the analysis. Nine veld zone types were identified in South Africa and dimensionless unit hydrographs were derived for each zone. The number of catchments represented in each zone ranged from 5 to 18. A co-axial diagram to estimate mean storm losses in the 9 zones was developed. SANRA (1986) recommend that in South Africa the unit hydrograph approach is a reliable method for catchments ranging in size from 15 - 5000 km². Bauer and Midgley (1974) developed a simple-to-apply lag-route method of design flood estimation in South Africa, based on the results of the unit hydrograph technique.

2.3.2. Continuous Simulation Modelling

Continuous simulation models attempt to represent the major processes which convert rainfall into runoff. Historical rainfall data or stochastic rainfall series are used to generate outflow hydrographs over long time periods and the simulated flow are subjected to standard frequency analysis techniques. Thus, model parameters determined using a relatively short period of calibration and verification can be used together with a long climate series to yield flood frequency estimates (Calver and Lamb, 1995). If the model parameters can be related to the catchment characteristics, then the model parameters can be transferred to similar catchments.

According to Reed (1999) and Houghton-Carr (1999), continuous simulation modelling for design flood estimation was still in the experimental and developmental stage at the time. Reviewing contemporary literature, Cameron *et al.* (1999) express the opinion that, although not fully proven, the use of continuous modelling for design flood estimation has resulted in encouraging output. The use of continuous simulation modelling is conceptually attractive in that a continuous soil water balance is maintained and hence the hydrological state of the catchment prior to each storm is implicitly determined. However, the number of variables to calibrate may be substantial (ASCE, 1997).

In a variation to continuous simulation modelling, Rahman *et al.* (1998) summarise a "runoff file" approach, where the outputs from continuous simulation for selected conditions are stored for subsequent use. This reduces the expertise required to configure the model and repeated model calibration by different users.

The use of historical rainfall and the continuous simulation of catchment soil moisture make the simplifying assumptions of event based modelling unnecessary (Cameron *et al.*, 1999). Reed (1999) refers to "whole catchment modelling" which integrates hydrological, hydraulic and various impact models. The data demands of continuous simulation modelling are of concern to Reed (1999), but he concedes that such an approach overcomes many limitations of the design event approach and the complications of the joint probability approach.

Schulze (1989) argues for a continuous simulation modelling approach to design flood estimation, because:

- long periods of record are necessary for accurate estimation of design values,
- long series of observed flood data are generally not available, often contain inconsistencies and are frequently both non-homogeneous and non-stationary,
- in comparison to runoff data, longer data sets of rainfall of better quality are usually available for most regions in South Africa, and
- the exceedance probability of floods is generally not related to the exceedance probability of rainfall, as assumed in simple event based models.

2.3.2.1. Advantages and Disadvantages

The advantage of a continuous simulation modelling approach is that a complete hydrograph is generated and not only a peak discharge (Reed, 1999). Rahman *et al.* (1998) refer to numerous publications and summarise the advantages of continuous simulation models as follows:

- No synthetic storms are required, as actual storm records are used and hence critical storm duration is not an issue.

- Antecedent moisture conditions (AMC) are modelled explicitly and hence any subjectivity in attempting to account for AMC is removed.
- The statistical analysis of output implies that the return period of the output is not assumed to be equal to that of the input rainfall.

In a similar vein, Boughton and Hill (1997) list the following advantages of such a CSM system:

- actual rainfall data from the area are used and not general regionalised design values,
- the use of a calibrated rainfall-runoff model avoids the needs for assumptions about losses, and
- sequences equal in length to the assumed return period of the probable maximum flood can be generated and hence no assumption regarding the shape of the distribution in this range is necessary.

Rahman et al. (1998) refer to numerous publications and summarises the disadvantages of continuous simulation models as follows:

- the difficulties in adequately modelling the soil moisture balance and obtaining input data at the required temporal and spatial scale,
- the loss of "sharp" events if the modelling time scale is too coarse,
- the extensive data requirements which result in significant time and effort to obtain and prepare the input data, and
- the expertise required to determine parameter values such that historical hydrographs are adequately simulated.

In summary, the advantages of continuous simulation models are the simulation of the complete hydrograph and continuous simulation of antecedent soil moisture conditions. These need to be weighed against the challenges of input data preparation, assigning values to model parameters and regionalisation (Houghton-Carr, 1999). The currently available increased computing power and sub-daily rainfall and flow data in digital form, enables the continuous simulation of hydrographs to become a standard technique for estimating design floods (Cameron *et al.*, 1999). In the application of continuous simulation models for design flood estimation, the requirement that consistent model parameterisations are necessary for both continuous flow series and flood frequency simulation, needs to be borne in mind (Cameron *et al.*, 1999).

2.3.2.2. Applications

Calver and Lamb (1995) illustrated the use of continuous simulation modelling for design flood estimation on 10 varied catchments in the UK, ranging in size from 1 to over 400 km². Two models with between 5 and 10 parameters were used and acceptable results were obtained. Calver and Lamb (1995) question if the models used were sufficiently robust and highlight the sensitivity of the results to the quality of the input data.

Daily rainfall records are generally longer and more abundant than streamflow records and, with the use of stochastic data generation techniques, are not limited to the length of the historical rainfall record. Boughton and Hill (1997) combined a daily rainfall generating model to generate long streamflow sequences with daily rainfall as input to a calibrated rainfall-runoff model. A procedure relating annual maxima peak flow rates to annual maxima daily runoff volume was utilised. The stochastic rainfall model attempts to simulate the annual maxima daily rainfalls to reflect the actual

distribution of observed annual maxima. The Australian Water Balance Model (AWBM) is an explicit water balance model, capable of operating at daily or hourly time steps and simulates three surface storages with different capacities to represent partial area runoff. A limitation to the continuous modelling system developed by Boughton and Hill (1997) is the need for concurrent rainfall and runoff records to calibrate the model and to estimate peak discharges as a function of runoff volume. The calibration of the stochastic rainfall model to annual maxima may compromise other characteristics of rainfall, e.g. wet-wet sequences which may influence the simulated runoff. No uncertainty is built into the parameters of the model and hence the simulated sequences only reflect the variability of rainfall. No reference is made to catchment size limitations and the application of the model on large heterogeneous catchments is not discussed.

The IHACRES model was used by Steel (1998) to simulate instantaneous peak flows dating back to the 1870s for 11 rivers in Scotland using long daily rainfall series. The low variability of the simulated flow in some rivers was attributed by Steel *et al.* (1999) to be the consequence of using a daily time step in the model. The importance of a longer simulated record for design flood estimation is illustrated by Steel *et al.* (1999).

Cameron *et al.* (1999) used TOPMODEL to derive the frequency distribution of extreme discharges by continuous simulation. They simulated runoff from a 10.6 km² catchment in Wales, UK, and showed that parameter sets for TOPMODEL could be found that satisfied both hydrograph and flood frequency simulation.

Blazkova and Beven (2000) demonstrated a methodology for the estimation of flood frequency characteristics for both small and large catchments, given limited hydrological information. The evaluation was carried within the Generalised Likelihood Uncertainty Estimation (GLUE) framework where information was used to constrain the parameter sets within the model. According to Blazkova and Beven (2000), this resulted in a realistic reflection of the uncertainty in the estimation. The TOPMODEL rainfall-runoff model was then used within the GLUE framework to estimate cumulative distributions of peak flows at higher return periods.

Calver *et al.* (2000) presented a method for estimating flood frequencies based on rainfall-runoff modelling undertaken on a continuous time basis that could be generalised in space such that application was possible at both gauged and ungauged sites. It was concluded from the study that, although the method is said to be generic, this generality is based upon a region being comparatively rich in the volume and quality of data resources. According to Calver and Lamb (2000) it is true to say that continuous simulation modelling approach is likely to be able to produce good site specific modelled time series as a basis for expressing flood frequency. A procedure for a continuous simulation modelling approach to design flood estimation is well advanced in the UK and, the method used is potentially capable of wider application if care is exercised to check the hydrological appropriateness of input used (Calver *et al.*, 2000).

In South Africa the use of the conceptual-physical, daily time step *ACRU* model (Schulze, 1995) has been used to estimate design peak discharges. For example, Smithers *et al.* (1995; 1997) applied the *ACRU* model on the 760 km² Lions and Mpofana tributaries of the Mgeni River. The location, catchment discretisation and schematic flow paths used in the study reported by Smithers *et al.* (1995; 1997) are shown in Figure 2.2. Of the two gauging stations for which observed data were

available for model verification, only data from gauging station U2H013 could be utilised, as the observed daily peak discharges at U2H007 did not exceed a threshold of $31.7 \text{ m}^3 \cdot \text{s}^{-1}$, presumably because the gauging structure was not designed to measure events exceeding this value. Examples of daily observed and simulated peak discharges at U2H013 are illustrated in Figure 2.3.

The design floods estimated using the General Extreme Value (GEV) distribution fitted to the annual maximum series extracted from the observed and simulated series at U2H013 are depicted in Figure 2.4. From results such as these it was concluded that a continuous simulation approach could reliably be used for design flood estimation in the study area.

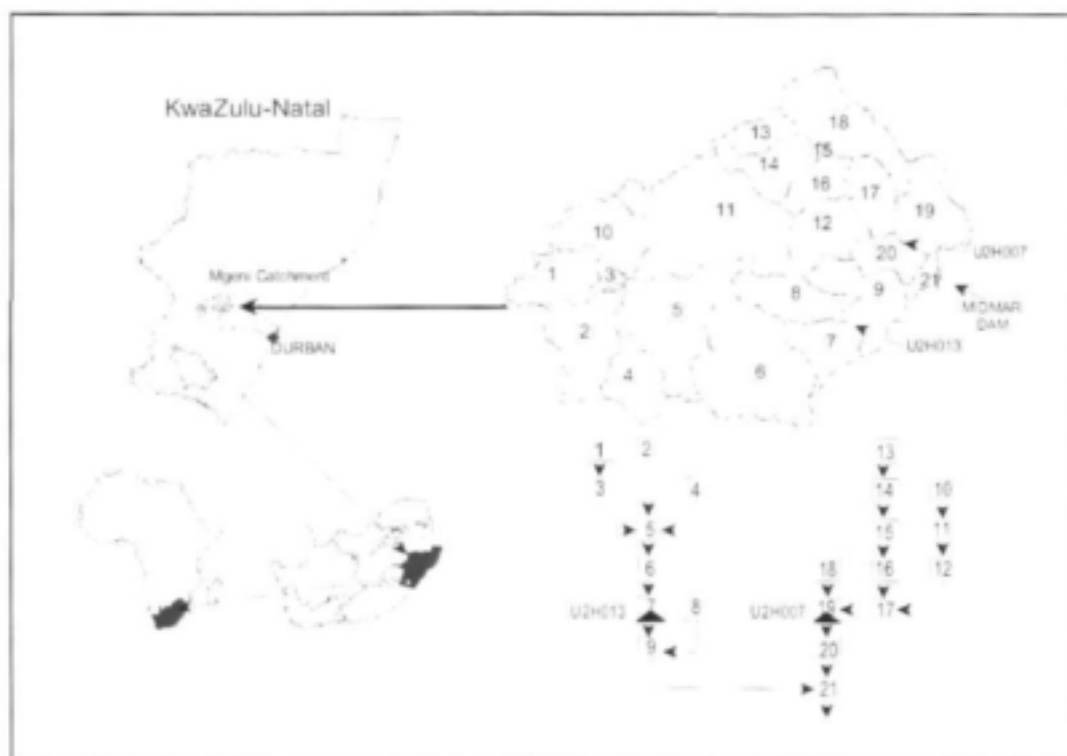


Figure 2.2 Location, catchment discretisation and schematic flow path (after Smithers *et al.*, 1997)

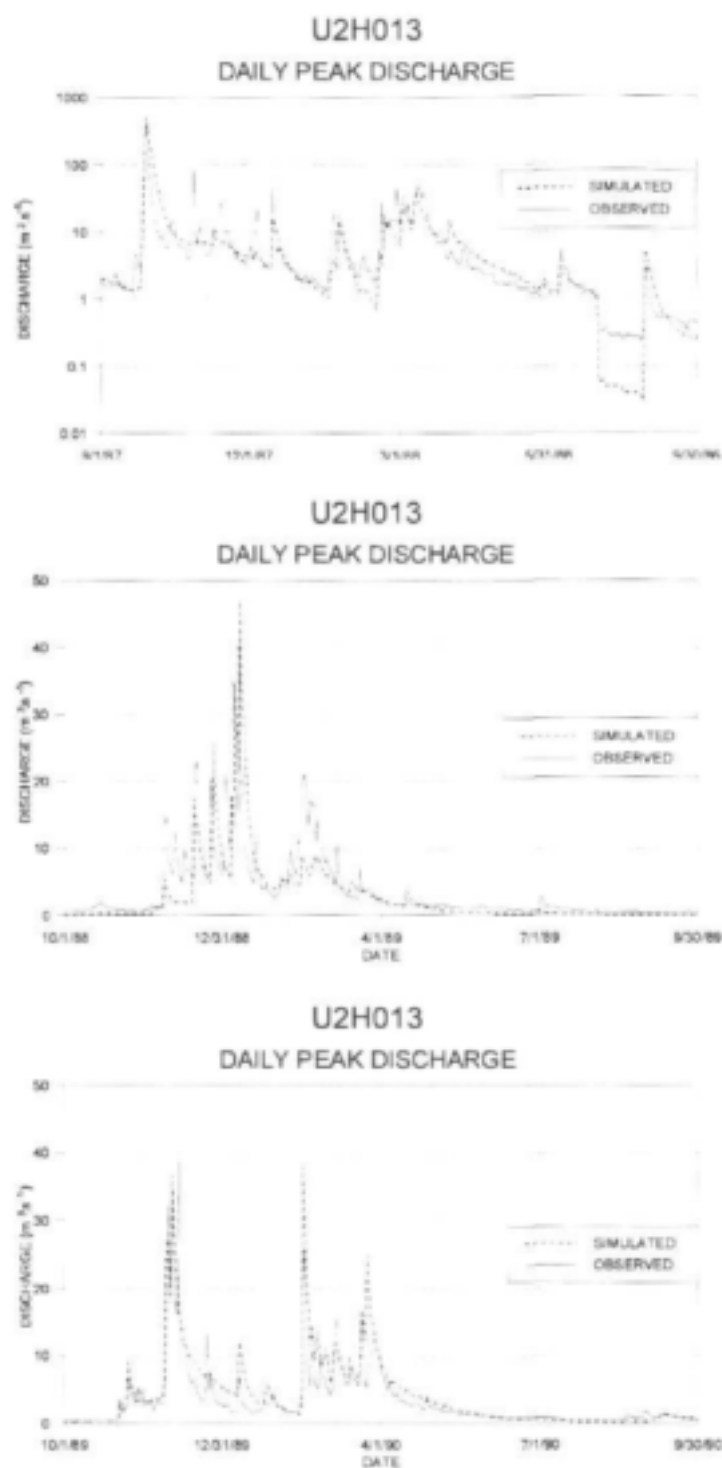


Figure 2.3 Examples of simulated (*ACRU* model) and observed daily peak discharge at U2H013 (after Smithers *et al.*, 1997)

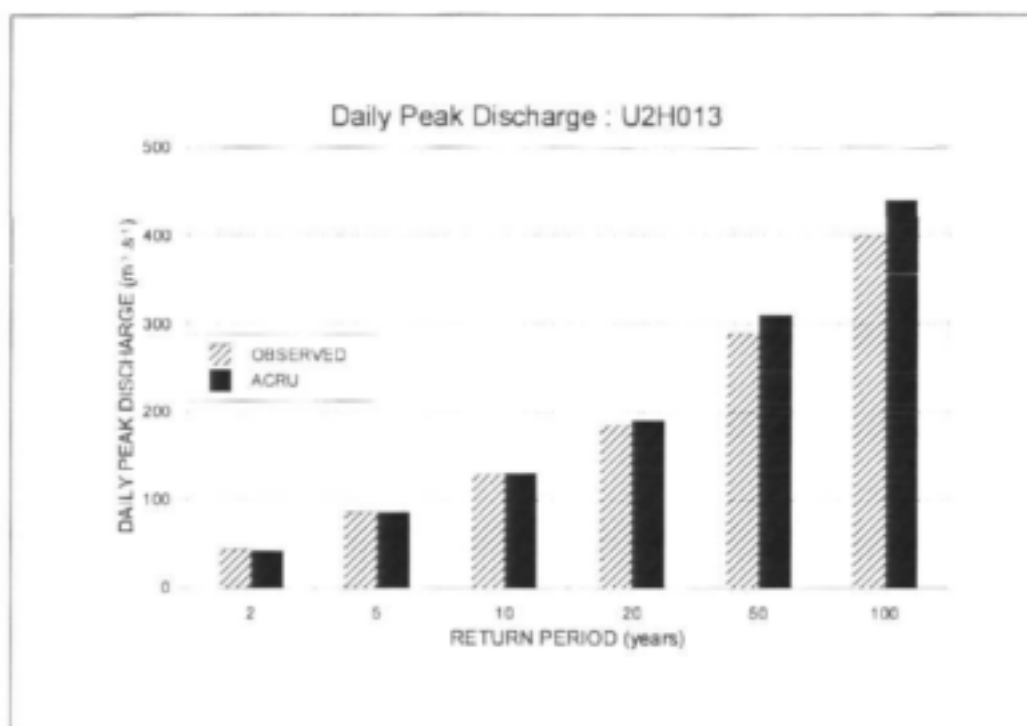


Figure 2.4 Design flood estimates for simulated (*ACRU* model) and observed data at gauging station U2H013 (after Smithers *et al.*, 1997)

More recently, a continuous simulation modelling approach was adopted by Smithers *et al.* (2000) to investigate the spatial variability, magnitudes and probabilities of the floods which occurred during the February 2000 floods in the Sabie River catchment. Again, the data from many gauging structures were either not available or not adequate for verification of floods simulated by the model, with overtopping of the weirs evident at many gauges. Examples of frequency analyses of observed and simulated daily peak discharges at gauging weirs X3H001 (173 km²) and X3H006 (766 km²) are shown in Figure 2.5. Peak discharges and return periods of the February 2000 floods estimated from the simulated series compared favourably with initial, hydraulically based assessments of the flood magnitudes made by Van Bladeren and Van der Spuy (2000).

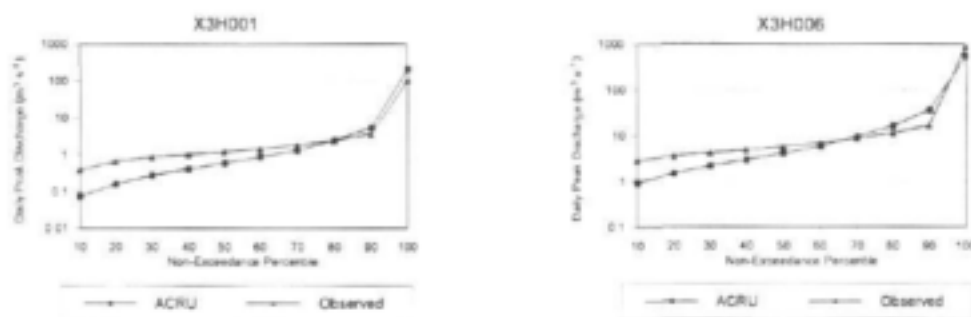


Figure 2.5 Frequency analysis of simulated and observed daily peak discharges at Gauging Weirs X3H001 and X3H006 (after Smithers *et al.*, 2001)

Smithers and Schulze (2003) presented research needs for design flood estimation in South Africa which included the following:

- A continuous simulation approach to design flood estimation should be further evaluated and developed. Such an approach overcomes many of the limitations of the design event approach and can accommodate current and projected future conditions in a catchment, such as anticipated land use or climate change. Limitations of the gauged flow data and changes in catchment conditions within the period of gauging may be overcome using this approach. It may be necessary to combine this approach with, for example, unit hydrographs to estimate the peak discharge. The output from a continuous simulation approach could be pre-run and packaged for hydrologically homogeneous regions/Quaternary Catchments to enable simple and rapid use by practitioners.
- Techniques for the temporal disaggregation and spatial estimation of daily rainfall need to be revised and refined.

The development of a continuous modelling system for design flood estimation in South Africa is detailed in the following chapter.

3. DEVELOPMENT OF A CONTINUOUS SIMULATION MODELING SYSTEM

As shown in Chapter 2, the *ACRU* model (1995) has been used as a continuous modelling system to estimate design floods in South Africa (Smithers *et al.*, 1995; Smithers *et al.*, 1997; Smithers *et al.*, 2000; Smithers *et al.*, 2001). The *ACRU* model has been verified widely on data from South Africa, Swaziland, Zimbabwe, Germany and the USA. Schulze (1995) presents 11 verification studies on various components of the *ACRU* model, both of outputs and internal state variables. Schulze and Pike (2004) list an additional 10 verification studies. Further verifications of model output are performed in this study. Given that the *ACRU* model has shown potential to be used to estimate design floods and was developed and verified in South Africa, it was selected as the continuous simulation model for this project and refinements to the model were made in this study.

This chapter contains a summary of the stormflow generation, peak discharge estimation and flood routing in the *ACRU* model as background to the developments made during this study. This include an investigation into the appropriate scale of input information and application of the model, the temporal disaggregation of daily rainfall, improved estimation of catchment rainfall and the development and evaluation of methods for flood routing in ungauged catchments. Also included is a study which relates antecedent soil moisture conditions prior to flood events and climate, which enables adjustments for antecedent moisture conditions (AMC) to be made in regions with limited hydrological data.

3.1. The *ACRU* Model

ACRU is described by Schulze and Smithers (2004) as a physical conceptual, multi-layer soil water budgeting model which operates at a daily time step, either at a point or as a distributed cell-type model. *ACRU* is multi-purpose, as it integrates soil water budgeting and runoff generation with, for example, crop yield estimates, dam yield analysis, irrigation demand and supply, and is multi-level in the sense that different levels of information may be input, dependent on the availability of data. A comprehensive overview of the historical development of the *ACRU* model is provided by Schulze and Smithers (2004).

3.1.1. Stormflow Generation

The partitioning of rainfall in the two layer soil water budget is depicted in Figure 3.1. Stormflow is generated using a modified SCS-based approach as shown in Equation 3.1

$$Q = \frac{(P - I_a)^2}{(P - I_a + S)} \quad \text{for } P > I_a \quad \dots \text{Eq. 3.1}$$

where

- Q = daily stormflow depth (mm),
- P = daily rainfall depth (mm), excluding intercepted rainfall,
- S = soil water deficit (mm) prior to the rainfall event which is computed for a critical response depth,

- I_a = initial losses (abstractions) prior to the commencement of stormflow, comprising of depression storage and initial infiltration (mm),
= c.S, and
- c = input coefficient of initial abstraction which has a default value of 0.2, but can vary from month-to-month dependent on vegetation and management practices (Schulze, 1995).

Baseflow is generated as a release from a "baseflow store", which accumulates percolation from the subsoil horizon. Stormflow is added to the baseflow component to simulate daily runoff from a catchment.

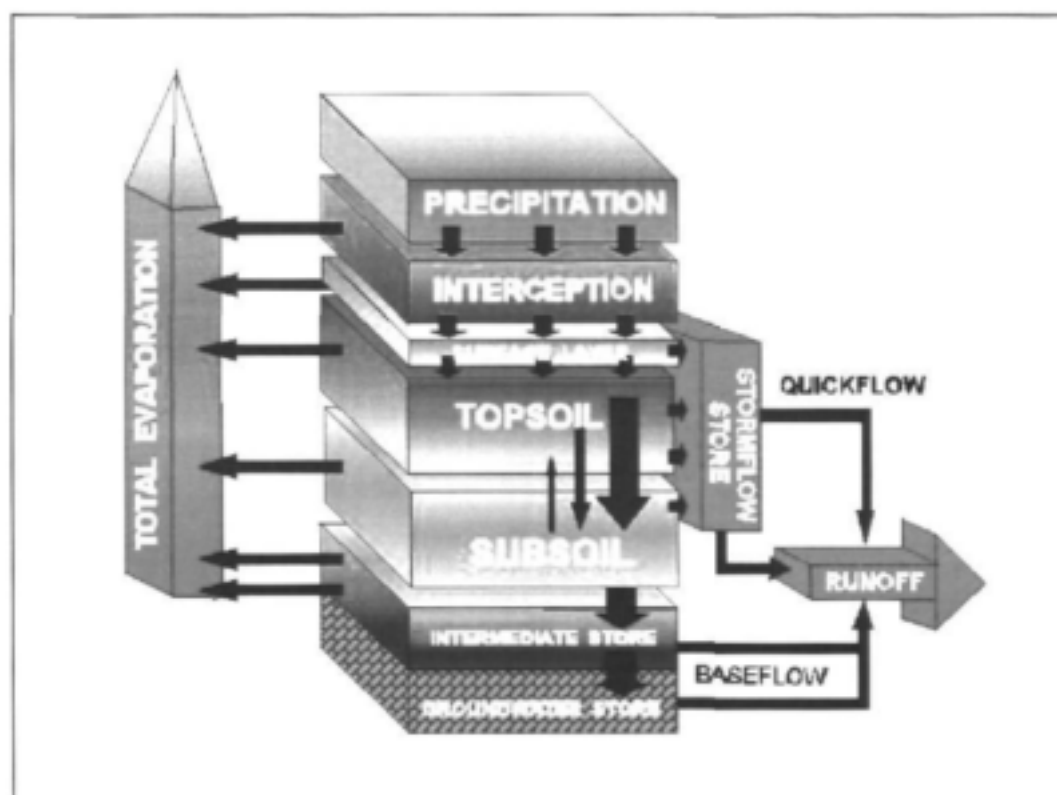


Figure 3.1 The ACRU agrohydrological modelling system: General structure (Schulze, 1995)

3.1.2. Peak Discharge Estimation

The calculation of peak discharge by SCS techniques is based on a triangular unit hydrograph concept. This unit hydrograph represents the temporal distribution of stormflow for an incremental unit depth of stormflow, ΔQ , occurring in a unit duration of time, ΔD . Peak discharge for an increment of time (ΔD) is defined by Equation 3.2

$$\Delta q_p = \frac{0.2083 \Delta Q \Delta}{\Delta D / 2 + L} \quad \dots \text{Eq. 3.2}$$

where

- Δq_p = peak discharge of incremental unit hydrograph ($\text{m}^3 \cdot \text{s}^{-1}$),
- A = catchment area (km^2),
- ΔQ = incremental stormflow depth (mm),
- ΔD = unit duration of time (h), used with the distribution of daily rainfall to account for rainfall intensity variations, and
- L = catchment lag (h), an index of the catchment's response time to the peak discharge.

As shown in Figure 3.2, to determine the hydrograph response to a given rainfall event, incremental hydrographs are superimposed according to the distribution of stormflow over time, as determined from the time distribution of rainfall intensity and the stormflow response characteristics of the catchment. Thus the daily rainfall input into *ACRU* is disaggregated within the model into sub-daily time steps using one of four regionalised synthetic rainfall distributions, shown in Figure 3.3, developed by Weddepohl (1988) and also used for design flood estimation based on adaptations for South Africa to the SCS model (Schmidt and Schulze, 1987).

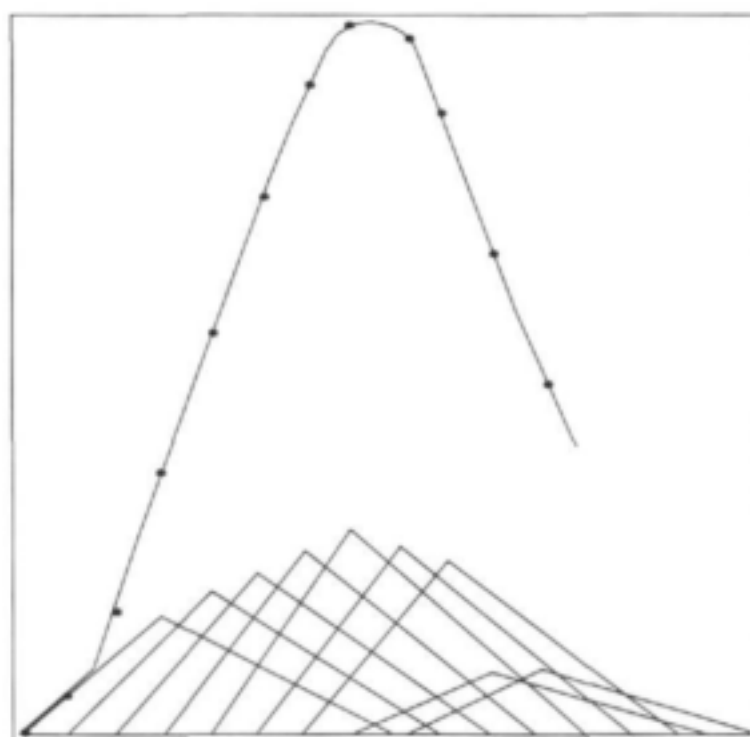


Figure 3.2 Superpositioning of incremental triangular unit hydrographs to generate the stormflow hydrograph (Smithers and Caldecott, 1993)

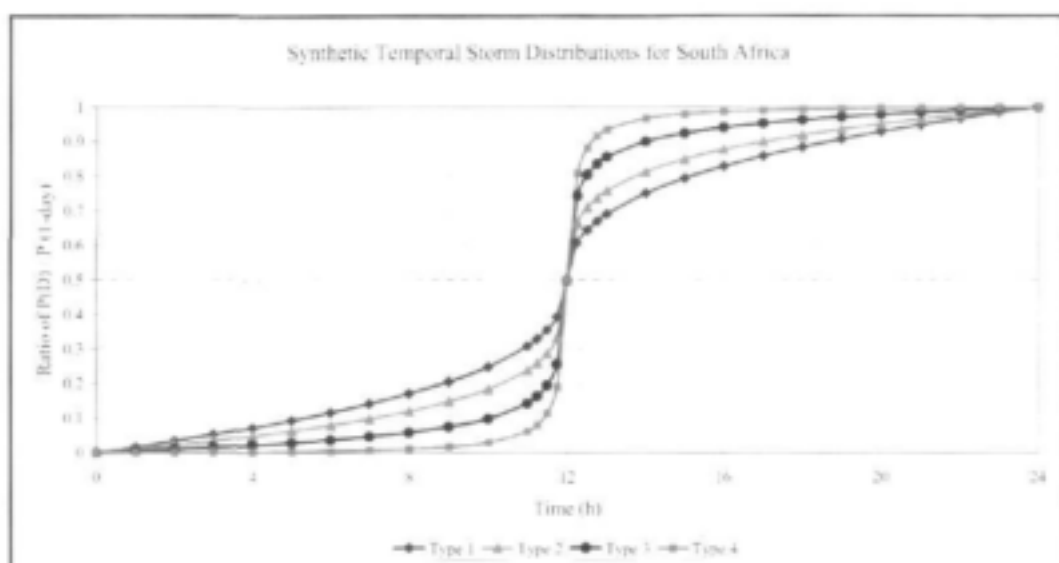


Figure 3.3 Synthetic temporal distributions for South Africa used in the *ACRU* model

3.1.3. Flood Routing

When operated in distributed mode, runoff generated from each sub-catchment may either be transferred instantaneously to the downstream sub-catchment or the flood routing routines may be invoked. A hydrograph is modified in two ways as it travels downstream through a river reach or reservoir, as shown in Figure 3.4. Hydrographs are routed through dams using the Storage-Indication (SI) method and the Muskingum-Cunge (MC) method of flood routing is utilised for hydrograph routing in river reaches (Smithers and Caldecott, 1995).

The SI method requires the user to input the surface area:volume relationship of the dam, the width and coefficient of discharge of the spillway and to select a routing time step to be used in the computations (Smithers and Schulze, 1995). Three options are available in the *ACRU* model to users to input the K and X parameters of the Muskingum method. The first input option is for single reach routing using non-varying user-input parameters, the second multiple reach routing using non-varying user-input parameters and the third option is for multiple reach routing using internally generated varying parameters. The internally generated parameters require the user to specify the cross-sectional shape and dimensions of a typical reach, Manning's roughness coefficient, and the length and slope of the reach (Smithers and Schulze, 1995). Details of an assessment of Muskingum-based flood routing methods and the estimation of flow characteristics using empirical relationships are contained in Section 3.6.

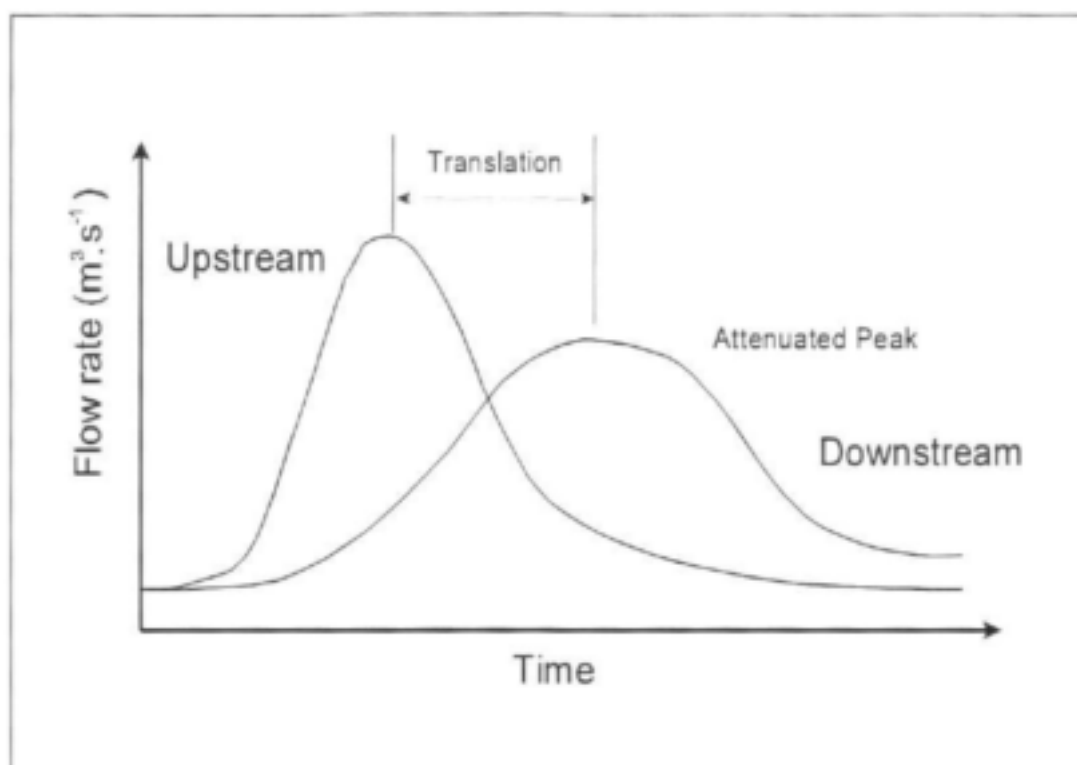


Figure 3.4 Hydrograph translation and attenuation (after Shaw, 1988)

3.2. Scale and Levels of Soil and Land Cover Information

According to Martina (2004) event-based or derived distribution approaches are not able to adequately follow the “physical chain” of hydrological processes in order to assess the role of each of the variables involved in rainfall-runoff processes, whereas a continuous simulation modelling approach aims at preserving the physical relationships between variables. A time series contains not only extreme events but also other events. Therefore, in terms of a CSM approach for flood estimation, a critical question which needs to be addressed is the “level” of representation that will preserve the “physical chain” of the hydrological processes, both in terms of scale of representation and level of description for the modelling process (Martina, 2004).

In the development of a CSM system in this study, the spatial resolution at which continuous simulation modelling is implemented is important in order to obtain realistic estimates of large flood events and design floods. It is therefore necessary to investigate the appropriate range of scales at which the CSM system could be applied to identify the appropriate levels of representation. Soil and land cover information also play an important role in a CSM as these are the prime regulators of a catchments response to rainfall, and therefore directly influence the hydrological response of a catchment. The *ACRU* model, which has been selected for use as a continuous simulation model in this study, can operate in lumped or distributed modes and soils and land cover information which are used as input to the model can be aggregated at different levels to represent spatial variability within a catchment.

The primary objective of this component of the study is to investigate the appropriate scale at which the CSM should be configured with respect to levels of spatial

disaggregation of a catchment and of soil and land cover information required to give optimum results i.e. to investigate the optimal level of catchment discretisation for the *ACRU* model used in this study. Simulations at Quaternary Catchment (QC) scale and sub-quaternary scale (sub-QC) with different levels of soils and land cover information have been undertaken in the Lions River catchment by Chetty *et al.* (2003) and further preliminary investigations undertaken in the Thukela catchment by Smithers and Chetty (2005) provides the basis for this study.

3.2.1. The Thukela Catchment Study Area

The Thukela catchment extends latitudinally from 27°E25'S to 29°E24'S and longitudinally from 28°E58'E to 31°E26'E, as shown in Figure 3.5. The catchment covers an area of 29 035.9 km² and forms the DWAF designated Thukela water management area. The Thukela River is the main river in the catchment and its major tributaries are the Little Thukela, Mooi and Bushmans Rivers which join from the southwest, and the Klip, Sundays and Buffalo Rivers flowing from the north. The Thukela river has its source in the Drakensberg mountain range in the west. The Drakensberg is a declared World Heritage Site which, in places, has altitudes exceeding 3000 m. The Thukela river flows eastward from a steep escarpment across low mountains of high relief, open hills of high relief and lowlands of low relief and thereafter through a deeply incised valley until it reaches the Indian Ocean approximately 85 km north of the city of Durban.

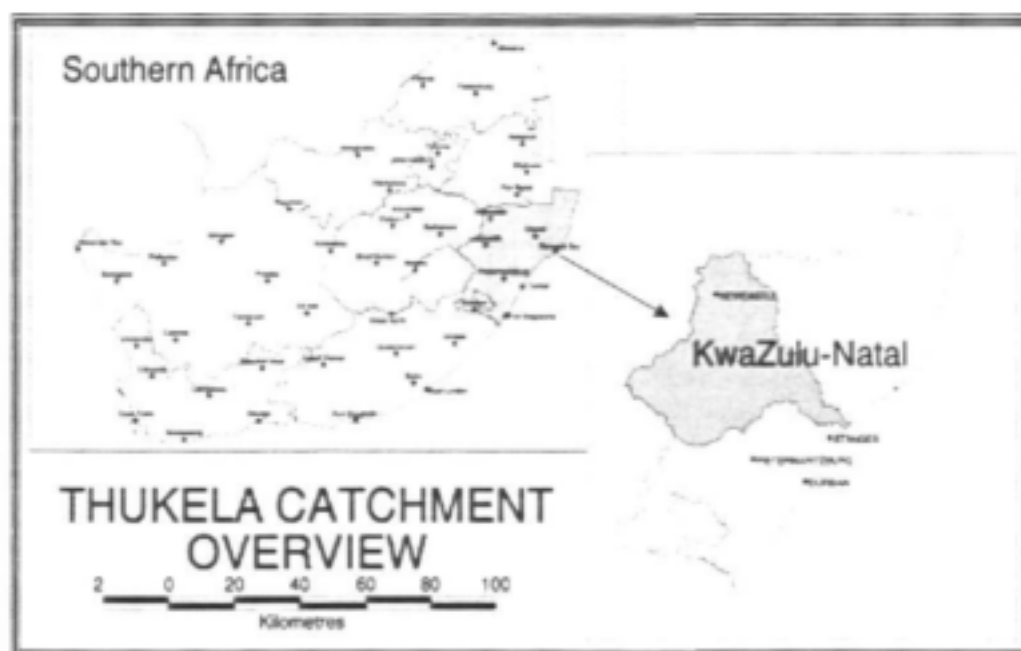


Figure 3.5 The Thukela catchment in KwaZulu-Natal, South Africa

South Africa has been divided into primary, secondary and tertiary catchments by the Department of Water Affairs and Forestry. The tertiary catchments make up the nineteen water management areas described in the country's national water resources strategy (DWAF, 2004). The Thukela catchment is one of the 19 Water Management Areas and at the fourth level of catchment delineation, the Thukela is divided into 86

QCs all of varying areas. From the range of catchment areas for the QCs in the Thukela Catchment, as shown in Figure 3.6, it is evident that the majority of the 86 QCs which constitute the Thukela catchment have catchment areas of less than 500 km².

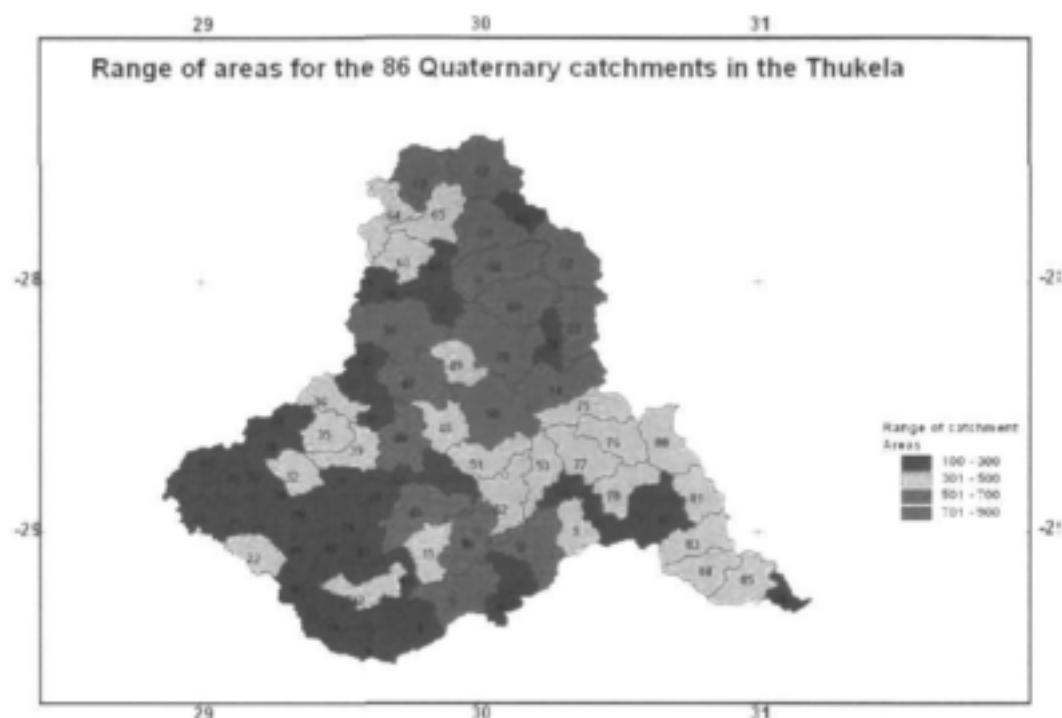


Figure 3.6 Range of areas (km²) for the 86 Quaternary Catchments in the Thukela Catchment

Schulze *et al.* (2005b) further divided the Thukela Catchment into 235 sub-quaternary catchments (sub-QCs). This delineation was mainly to accommodate the diverse hydrological response within many of the QCs and also to explicitly represent various heterogenic factors which included altitude, soils, topography and vegetation, channel based factors such as the locations of flow gauging weirs and dams, the locations of environmental flow requirement sites, and political history which resulted in degraded vegetation (Schulze *et al.*, 2005a).

3.2.2. Research Methodology

The research strategy adopted for this component of the study was to select upstream QCs, which included a range of sub-catchment areas, each with a selected representative rainfall station located either within or nearby the sub-catchment and with a reliable flow gauging station at the outlet of the catchment. Each of the selected QCs were then configured using different spatial scales i.e. lumped (single entity or whole QC), sub-catchments as delineated by Schulze *et al.* (2001), and hydrological response units (HRU) modelled using catchment specific land cover. For each of the spatially configured catchments, different levels of soils information were used as input. This included area weighted values of soils information and modal values (soil with highest frequency of occurrence within the specified area). For lumped

catchments, information on the dominant land cover was used as input. For sub-QCs the information on the dominant land cover in each sub-catchment was used and for the HRU identification four major categories of land use were identified viz. thicket and bushveld, cultivated crops, forestry and grasslands. The scenarios at the different spatial scales and with different levels of soils and land cover information are summarised in Table 3.1.

Table 3.1 Soils and land cover information used for the simulations of the scenarios for different spatial scales

Scenario	Soils Information	Land Cover Information	Sub-catchments / HRU's
Lumped	Modal for whole catchment	Modal for whole catchment	Lumped
MA	Area weighted per sub-catchment	Modal per sub-catchment	Sub-catchments
HA	Area weighted per sub-catchment	Catchment specific	Hydrological response units
HM	Modal per sub-catchment	Catchment specific	Hydrological response units

The scenarios were named, firstly, according to the spatial discretisation and, secondly, to differentiate between the levels of soil information used. For example, MA implies modal land cover and area weighted soil and HA implies HRU and area weighted soil. In the selected QCs which Schulze *et al.* (2001) had further delineated into sub-QCs, more than one driver rainfall station was used in the simulations. Thus some scenarios are referred to as "HA1d" where "1d" implies that one driver station was used and some scenarios as "HA2d", where "2d" indicates that two driver stations were used in the simulations.

Upstream or external catchments were selected initially according to the availability of reliable streamflow and rainfall data and thereafter according to the size of the catchment. In practice it was difficult to select catchments from a wide range of catchment areas which met all of the criteria required. This was due to the Thukela Catchment being an operational catchment in which significant water transfer schemes and irrigation take place, as well due to the operation of large dams in some catchments.

Three QCs ranging in size from 129 km² to 544 km² were selected as test catchments for this component of the study, as shown in Figure 3.7. This range of catchment areas was deemed to be sufficient as most QCs in the Thukela Catchment are less than 500 km². It was postulated that the results of this component of the investigation would be applicable for larger QCs.

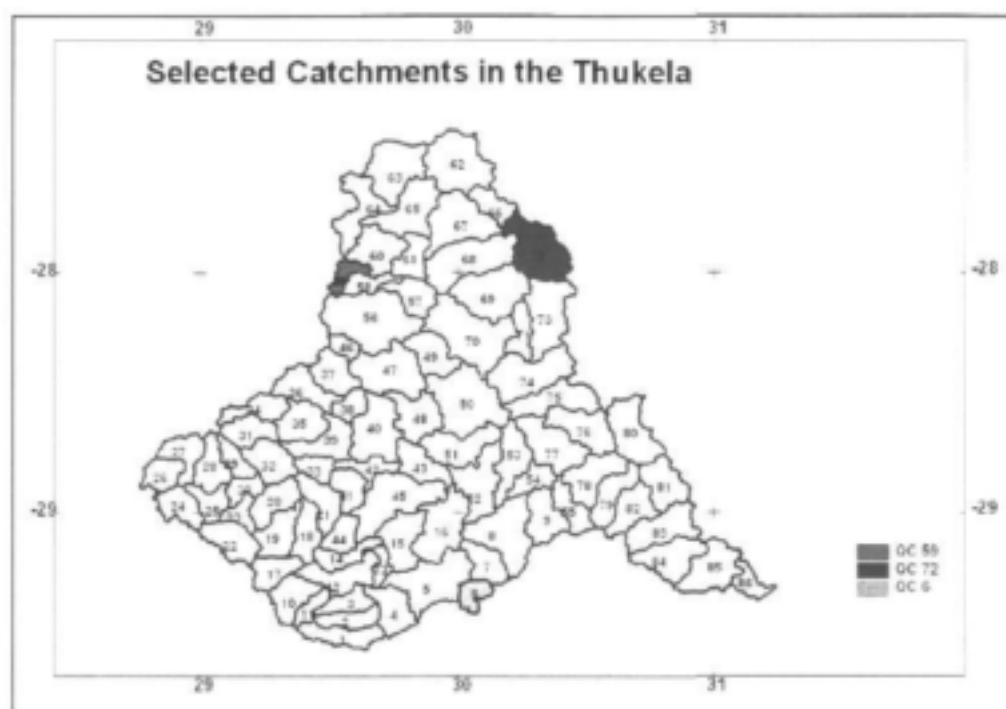


Figure 3.7 Selected Quaternary Catchments in the Thukela Catchment

For each of the selected catchments, the different scenarios were configured as detailed in Table 3.1 and input menus (input data and information files) for the *ACRU* model were created for each of the scenarios. The *Calc_pptcor* (Schulze and Pike, 2004) suite of programmes were used to select the most representative rainfall station, termed the driver rainfall station, and to compute the precipitation correction factors that are used to adjust the gauged daily rainfall data to represent catchment rainfall. Soils information was obtained from the ISCW soils maps (SIRI, 1987) at the 1: 50 000 resolution and which have been translated into *ACRU* variables by Schulze and Pike (1995). Land cover information was obtained from the National Land Cover Database (CSIR, 1999) at a resolution of 1: 250 000 and which has been translated into *ACRU* variables by Schulze (2001). For the purpose of this investigation both area weighted and modal soils information were computed for catchments, both at quaternary and sub-quaternary scale.

In order to compare the effects of soil and land cover information at the different spatial resolutions on runoff depths, the scenarios summarised in Table 3.1 were configured for each of the selected catchments listed in Table 3.2. The period of simulation was from 1950 to 1999.

Table 3.2 Information for selected catchments

QC	Area (km ²)	Gauging Weir	No. of sub-QCs	Total No. of HRUs per QC	Dam
59	129	V3H007	3	12	No
6	152	V2H016	1	4	Yes
72	544	V3H011	2	8	Yes

3.2.3. Results

The results for QCs 59, 6 and 72 are shown in Figure 3.8 to Figure 3.13. In order to meet the objectives of this component of the study, accumulated simulated values for all scenarios and accumulated observed runoff are compared for each selected QC for periods when non-missing observed data are available and considered to be reliable.

The accumulated totals as well as the Root Mean Square Error (RMSE) of daily values were calculated for each scenario. In addition, a frequency analysis of simulated and observed daily runoff depths for all scenarios was performed on values which are greater than zero.

Accumulated streamflows over the period 1958 to 1999 are shown in Figure 3.8 for QC 59. The accumulated simulated values compare well with the accumulated observed data indicating that for the given period the *ACRU* model is performing well. It is important to note that there are differences between each of the scenarios. The differences between HA1d with HA3d highlight the effect when more than one driver rainfall station is used and comparing HA1d with HM1d and HA3d with HM3d shows the differences in modelling with area weighted versus modal soils information. The differences between the use of a single modal land cover and HRUs is evident when comparing, for example, scenarios HA3d with MA3d. The results shown in Figure 3.8 indicate that, from 1974 onwards, the lumped scenario results in over-simulation when compared to the observed flows and therefore using HRUs is more appropriate.

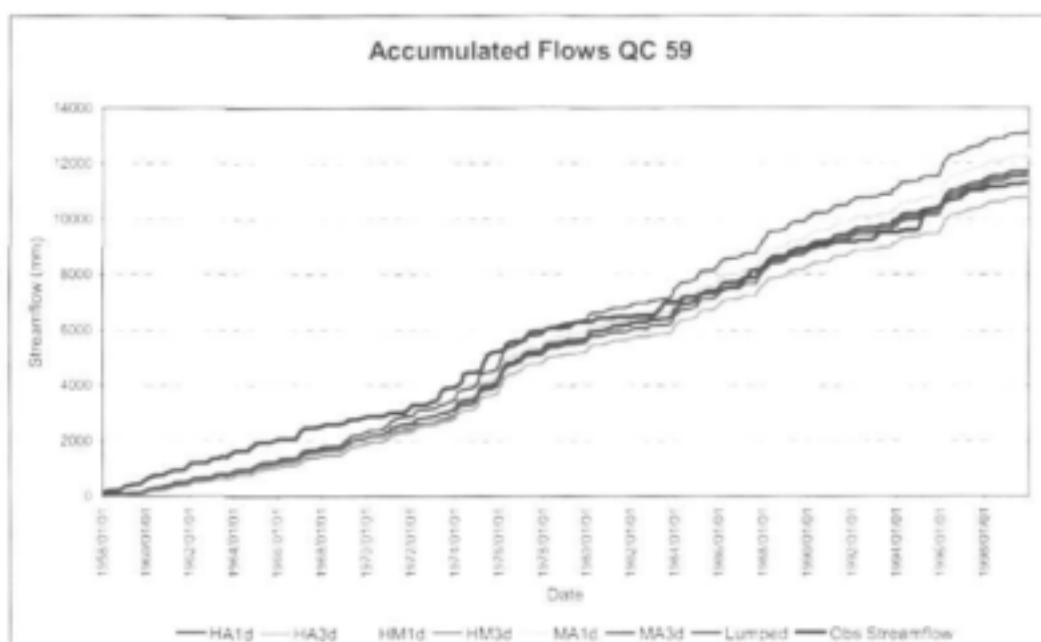


Figure 3.8 Accumulated observed and simulated streamflows for all scenarios in QC 59

It is however difficult to judge from a visual inspection which scenario best represents the observed data and the accumulated totals and RMSE for the simulated and observed daily values are summarised in Table 3.3. It is evident from the results presented in Table 3.3 that, the total accumulated depth for the lumped scenario is much larger than both the accumulated observed depth as well as the accumulated depths for all the other scenarios. This is further emphasised by the largest RMSE of 2.64 mm obtained for the lumped scenario. The results also indicate that scenarios which use area weighted soils information perform better than scenarios which use modal soils information. For example, the RMSE of HA1d is less than that for HM1d, and the RMSE of HA3d is less than the RMSE for HM3d. Considering both total accumulated flow depths and the RMSE values, the best scenario in QC 59 is HA3d, where the accumulated simulated and observed flow volumes are similar and HA3d resulted in the smallest RMSE of 2.32 mm.

The results of frequency analyses performed on daily observed and simulated runoff depths for all scenarios are shown in Figure 3.9. The *ACRU* model simulates the observed flows well even though there is slight under simulation of the larger events.

Table 3.3 Total accumulated streamflow depths and the RMSE of the observed and simulated values for QC 59

Scenario	Spatial Representation	Level of Land Cover	Level of soils information	No. of Driver Rainfall Stations	Total Accumulated Streamflow (mm)	RMSE (mm)
Observed					11314	
HA1d	12 HRUs	Catchment specific	Area weighted	1	11586	2.47
HA3d	12 HRUs	Catchment specific	Area weighted	3	10841	2.32
HM1d	12 HRUs	Catchment specific	Modal	1	12489	2.59
HM3d	12 HRUs	Catchment specific	Modal	3	11669	2.42
MA1d	3 Sub-catchments	Modal	Area weighted	1	12326	2.48
MA3d	3 Sub-catchments	Modal	Area weighted	3	11802	2.34
Lumped	1 QC	Modal	Modal	1	13162	2.64

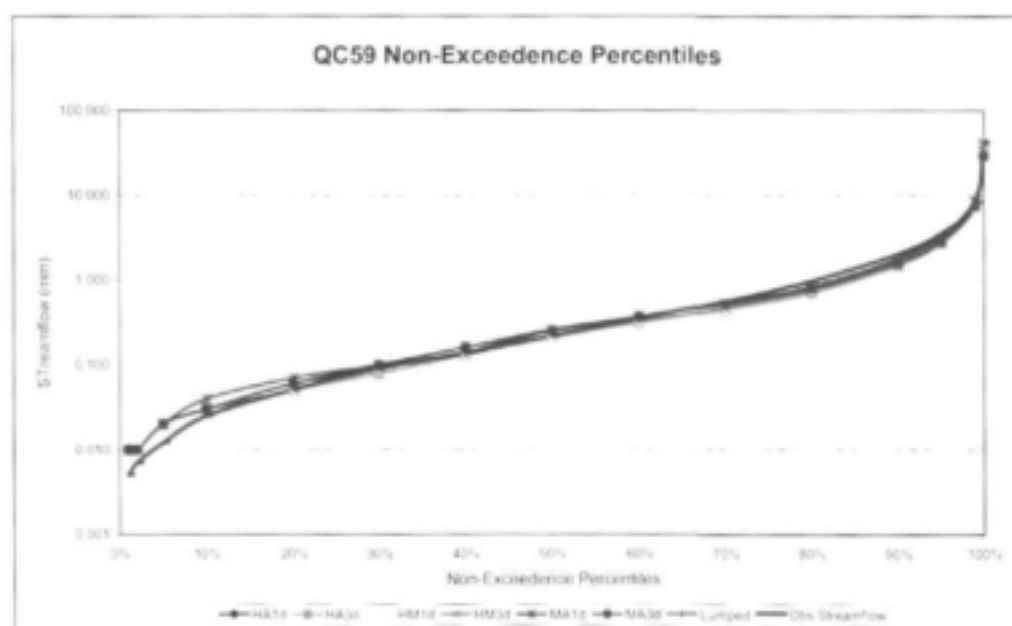


Figure 3.9 Frequency analysis of simulated and observed daily runoff depths for QC59

The accumulated simulated and observed flows in QC6 for the period 1963 to 1999 are shown in Figure 3.10. Only three scenarios were modelled for this QC because it was not divided into sub-QCs by Schulze *et al.* (2001). It is evident from Figure 3.10 that there is some over-simulation by the *ACRU* model for all scenarios. This discrepancy could be attributed to the relatively large Craigieburn dam which is present in the catchment. From the results contained in Figure 3.10 it is evident that there are differences between the scenarios modelled. For example, the lumped scenario over-simulates more than all the other scenarios considered.

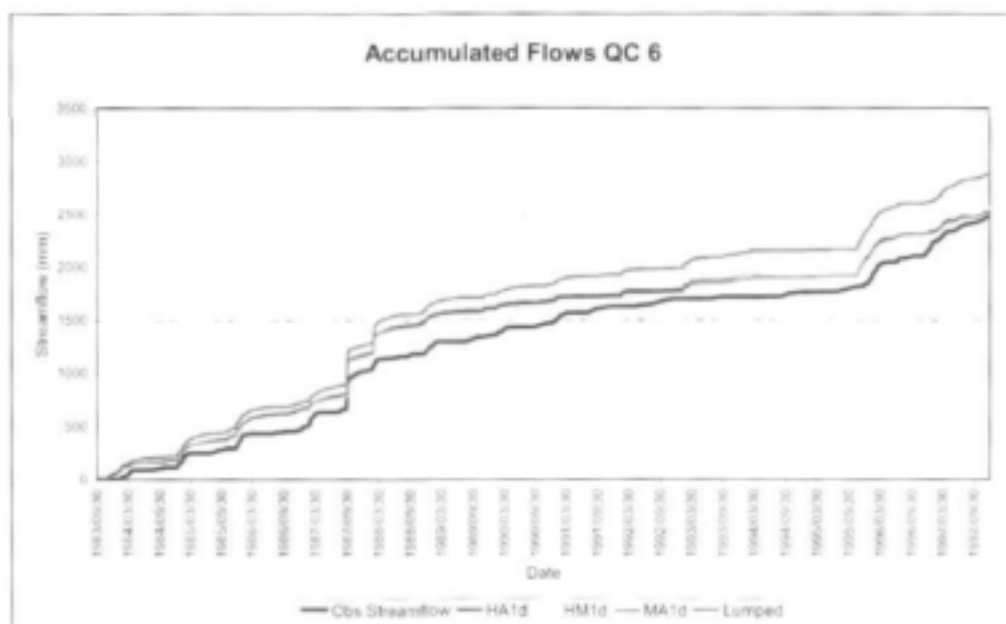


Figure 3.10 Accumulated flows for QC6 for all scenarios

The trends are summarised in Table 3.4 which contains the accumulated totals as well as the RSME of daily values for all scenarios in QC 6. There is a small difference in accumulated totals for scenarios HA1d and HM1d. However, the RMSE shows that HM1d has a larger error value implying that in QC6 the use of area weighted soils yield improved simulations. The results for scenarios HA1d and MA1d are very similar and this is reflected in the results from the frequency analyses shown in Figure 3.11 and in the total accumulated streamflows and RMSE contained in Table 3.4. This could be due to the fact that QC 6 is relatively homogenous and hence was not divided into sub-QCs. Both scenarios HA1d and MA1d simulated the observed values well in terms of both total accumulated streamflow depths and low RMSE values.

Table 3.4 Total accumulated streamflow depths and the RMSE of the observed and simulated values for QC 6

Scenario	Spatial Representation	Level of Land Cover	Level of soils information	No. of Driver Rainfall Stations	Total Accumulated Streamflow (mm)	RMSE (mm)
Observed					2484	
HA1d	4 HRU's	Catchment specific	Area weighted	1	2551	1.73
HM1d	4 HRU's	Catchment specific	Modal	1	2555	1.78
MA1d	1 QC	Modal	Area weighted	1	2521	1.73
Lumped	1 QC	Modal	Modal	1	2873	1.73

The results of frequency analyses of observed and simulated daily runoff depths for QC 6 are presented in Figure 3.11. The model slightly over-simulates the larger events which could be attributed to the large dam in the catchment. There are larger differences for the smaller events which could be due to erroneous observed low flow measurements or that the low flows are not simulated adequately by the *ACRU* model.

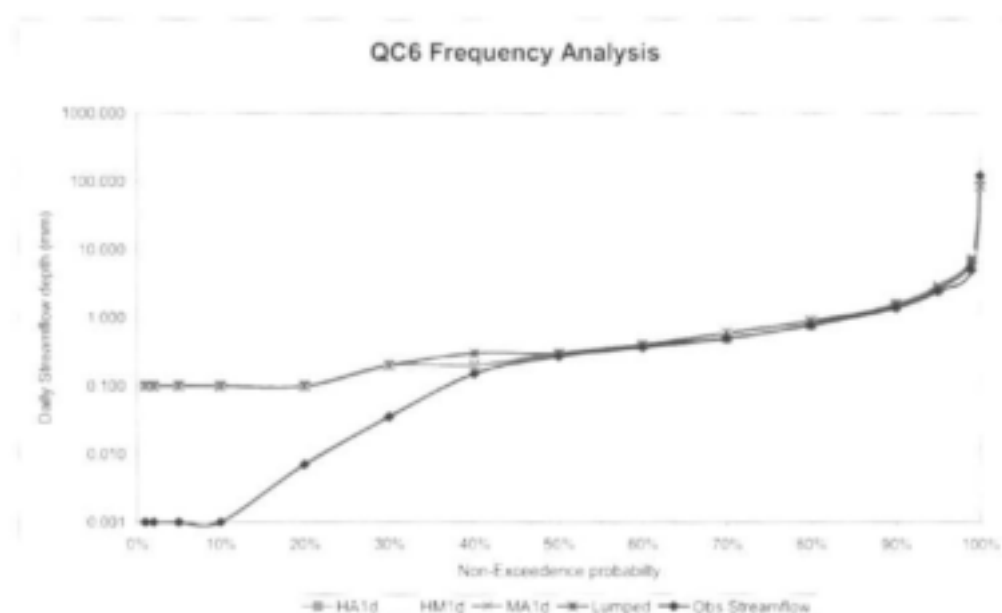


Figure 3.11 Frequency analysis of simulated and observed daily runoff depths for QC6

The accumulated simulated and observed streamflow depths for all scenarios for QC 72 are presented in Figure 3.12 and, the accumulated totals of streamflow depths for simulated and observed values as well as the RMSE for all scenarios are contained in Table 3.5. These results indicate that the lumped scenario and scenario MA1d are very similar and both over-simulate the observed runoff depth. These two scenarios have the largest accumulated total streamflow depths and RMSE values. Since QC6 was not further discretized into sub-QCs it is expected that these two scenarios will produce similar results.

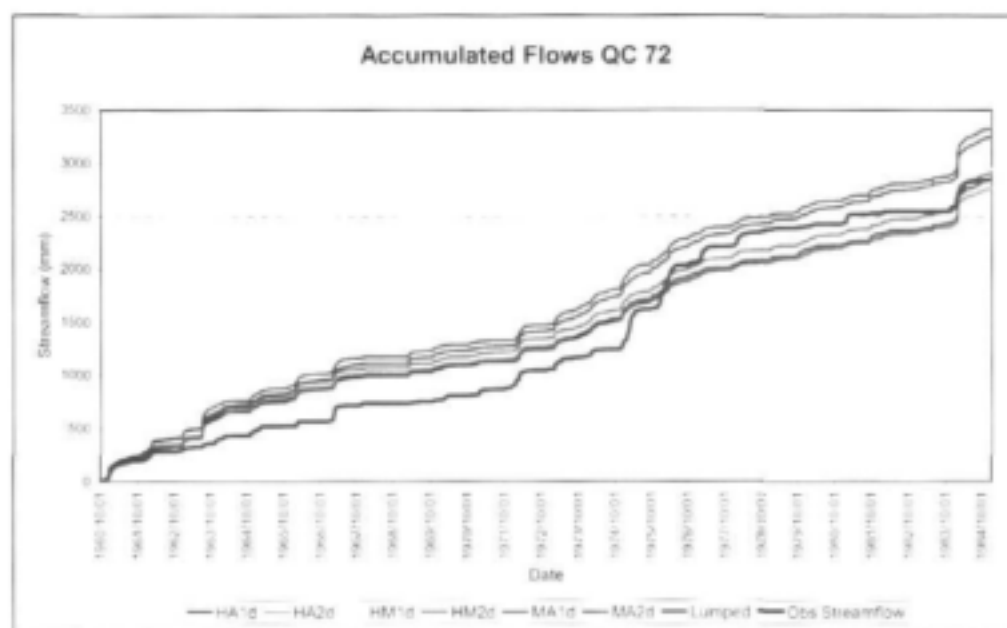


Figure 3.12 Accumulated flows for QC72 for all scenarios

Increasing the number of driver rainfall stations made little difference to the simulations since the RMSE values, as shown in Table 3.5, for the HA1d and HA2d and HM1d and HM2d scenarios are similar. Simulations obtained using area weighted soils information performed better in terms of accumulated totals and RMSE values, than simulations using modal soils information. Comparing total accumulated streamflow depths and the RMSE values for scenarios HA1d and MA1d illustrates that modelling as HRUs results in improved simulations. The best results were obtained for Scenario HA1d which had the lowest RMSE of 7.68 mm and the accumulated total streamflow depth closest to the observed total.

Table 3.5 Total accumulated streamflow depths and the RMSE of the observed and simulated values for QC 72

Scenario	Spatial Representation	Level of Land Cover	Level of soils information	No. of Driver Rainfall Stations	Total Accumulated Streamflow (mm)	RMSE (mm)
Observed					2796	
HA1d	8 HRUs	Catchment specific	Area weighted	1	2763	7.66
HA2d	8 HRUs	Catchment specific	Area weighted	2	2762	7.66
HM1d	8 HRUs	Catchment specific	Modal	1	2946	7.69
HM2d	8 HRUs	Catchment specific	Modal	2	2902	7.69
MA1d	2 Sub-catchments	Modal	Area weighted	1	3326	7.71
MA2d	2 Sub-catchments	Modal	Area weighted	2	2842	7.68
Lumped	1 QC	Modal	Modal	1	3247	7.70

The results of frequency analyses of the observed and simulated streamflow depths for all scenarios in QC 72 are shown in Figure 3.13. It is evident from Figure 3.13 that the *ACRU* model generally simulated the observed volume well, but the larger events were slightly under-simulated.

The scenario which resulted in the “best” simulations in all three catchments are presented in Table 3.6. It is evident from the results presented that in all three QCs the scenario in which QCs are divided into HRUs and area weighted soils information was used resulted in the best simulations.

Table 3.6 Summary of Results for QCs 59, 6 and 72

QC	Area (km ²)	Best Scenario
59	129	HA3d
6	152	HA/MA
72	544	HA1d

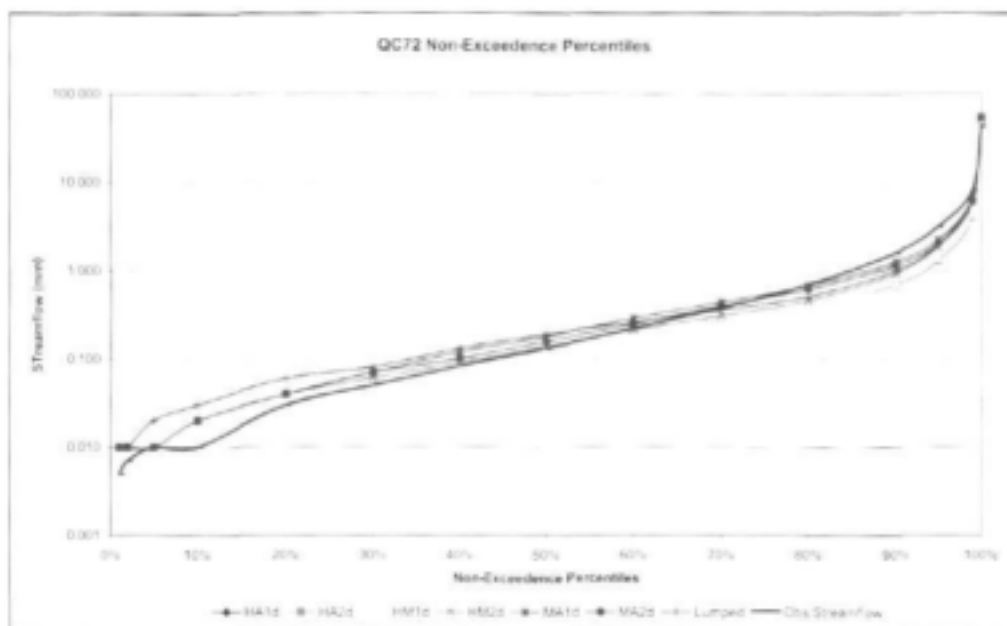


Figure 3.13 Frequency analysis of simulated and observed daily runoff depths for QC72

3.2.4. Discussion and Conclusions

The primary objectives of this component of the study was to investigate the appropriate scale at which the CSM should be configured for the Thukela Catchment and the levels of soil and land cover information required to give optimum results i.e. to investigate the optimal level of catchment discretisation and aggregation of input information for the *ACRU* model configured for the Thukela Catchment. The methodology adopted to perform this investigation required relatively un-impacted upstream catchments which had adequate and reliable data. This became a limitation to the study as not many catchments were un-impacted and which had long periods of record. Three QCs were selected with catchment areas ranging from 129 to 544 km². The majority of QCs within the Thukela catchment have catchment areas less than 500 km² and it was thus assumed that the results obtained for the selected QCs will be valid for all QCs in the Thukela Catchment.

The results obtained from the investigations in the QCs indicate that:

- QCs should not be modelled as lumped entities and should be discretised to HRUs as the best simulations were obtained using HRU scenarios.
- Using area weighted soils information yields better simulations and is therefore more appropriate to use than modal soils information.
- Modelling using more than one driver rainfall station per sub-QC yielded better results in QC 59 than modelling using a single driver rainfall station with different precipitation correction factors, but the use of additional rainfall stations in QC 72 did not make much difference. This could be attributed to the fact that there were not many reliable and representative rainfall stations in the area.

The findings obtained in this study concur with investigations done by Chetty *et al.* (2003) where catchment discretisation, appropriate levels of soil and land cover

information as well as the use of more than a single driver rainfall station per QC was investigated in the 353 km² Lions River catchment.

The recommendations from these results are that the optimum level of catchment discretisation in the Thukela Catchment should include sub-quaternary catchments where appropriate, and all catchments should be modelled using HRUs with area weighted soils information. Different driver rainfall stations for each of the sub-catchments which constitute a QC should be used where representative raingauges are available.

3.3. Daily Rainfall Disaggregation

Engineers and hydrologists involved in the design of hydraulic structures, such as dams, bridges and culverts, need to accurately assess the frequency and magnitude of extreme hydrological events. Current techniques for design flood estimation in South Africa need to be updated, regional approaches need to be evaluated and new techniques have to be developed and applied (Smithers and Schulze, 2001). One input that would improve rainfall-runoff modelling, and hence the estimation of design floods, is to account for the temporal distribution of rainfall.

The temporal distribution of rainfall, viz. the distribution of rainfall intensity during a storm, is an important factor affecting the timing and magnitude of peak flow from a catchment and hence the flood-generating potential of rainfall events (Weddepohl, 1988). It is also one of the primary inputs into hydrological models used for the design of hydraulic structures. The temporal distribution of rainfall events may be influenced by many factors that need to be reflected in design temporal distributions. These factors include, *inter alia*, location, storm duration, storm depth and season of storm occurrence (Hoang *et al.*, 1999).

The intensity distribution of a storm may be estimated by the use of a temporal distribution curve, which may be synthetically derived or obtained from observed hyetographs (Chow *et al.*, 1988). The use of temporal distributions is usually applicable to event-based models, such as the SCS-SA design flood estimation model (Schulze *et al.*, 1992). However, temporal distributions can also be used in rainfall disaggregation approaches (Boughton, 2000).

Rainfall disaggregation refers to producing high-resolution data that can be aggregated to give values equal to observed coarser-scale totals. The use of high-resolution rainfall data inherently accounts for the temporal distribution of rainfall intensity. This is because the incremental time-steps are small enough, i.e. hourly or sub-hourly, so as to represent different intensities. High-resolution rainfall data are often required as input into continuous simulation hydrological models. It is important to note that disaggregation is not synonymous with downscaling. Downscaling aims to produce finer scale time-series with the required statistics, like disaggregation, but do not necessarily add up to any coarser-scale totals (Koutsoyiannis, 2003). Downscaling is, in particular, used for hydrological applications of general circulation models.

Continuous simulation hydrological models are important tools when analysing complex hydrologic or hydraulic problems where issues need to be investigated at different timescales, for example in flood prediction and the modelling of water quality (Mikkelsen *et al.*, 1998). These models require detailed rainfall data, viz. hourly or sub-hourly. The advantage of such a time-series is that they reflect all relevant rainfall characteristics from peak intensities with short duration to variations

in annual rainfall (Mikkelsen *et al.*, 1998). However, data are generally only widely available at more aggregated levels of the model time-step, such as daily. Koutsoyiannis and Onof (2001) note that in many countries, the number of raingauges providing hourly or sub-hourly resolution data is smaller than the number of daily gauges by about an order of magnitude. This situation reflects a general relative paucity of rainfall data for timescales of one hour or less, both in the number of gauges and length of the recorded series (Koutsoyiannis and Onof, 2001). This, too, is the case in South Africa where it is reported that there were 172 recording gauges with at least 10 years of breakpoint data (Smithers and Schulze, 2000b), compared to 1806 daily rainfall stations with at least 40 years of data (Smithers and Schulze, 2000a). The need for a model to disaggregate daily rainfall into a sequence of individual storms of finer timescale cannot be overemphasised (Gyasi-Agyei, 1999).

3.3.1. The Disaggregation Model

The daily to hourly disaggregation model used and modified in this component of the study is based largely on the work done by Boughton (2000). The details of the model developed by Boughton (2000) are described in this section and changes to the methodology developed by Boughton (2000) are highlighted.

3.3.1.1. Structure of the Model

The model is comprised of 4 main parts:

- The distribution of the fraction of the daily total, R , that occurs in the hour of maximum rainfall.
- For each value of R there is an average set of values for the other 23 hourly fractions of the daily total.
- Given the 24 fractions from above, the values are clustered to maintain the observed average highest 2-hour, 3-hour, 6-hour and 12-hour fractions.
- These clusters are then arranged into random patterns so as to reproduce the possible variations in daily temporal patterns while retaining the above mentioned statistics.

3.3.1.2. Distribution of R

The primary part of the disaggregation model is the fraction, R , of the daily rainfall total that occurs in the hour of maximum rainfall. A value of $R = 1.0$ indicates that all of the rainfall on the day fell in a single hour. This is the upper limit of R and is the boundary of non-uniformity. Completely uniform rainfall throughout a day would yield $R = 0.04167$ (i.e. $1/24$ of the daily total). This is the lower limit of R .

An example of a single day's rainfall in hourly increments at Raingauge N23 at Ntabamhlope in the KwaZulu-Natal (KZN) midlands is shown in Figure 3.14. The daily rainfall total was 84.4 mm and the hour in which the most rainfall fell contained 40.9 mm. This yields a ratio of $R = 40.9/84.4 = 0.48$ for the day.

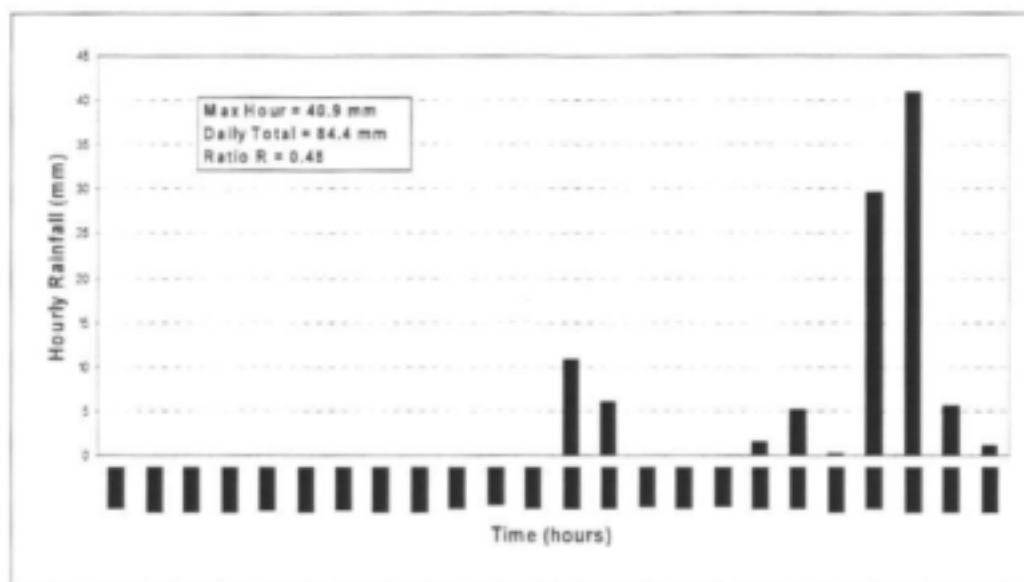


Figure 3.14 Example of a single day's hourly rainfall at Ntabamhlope

If the ratio R is determined for all days with 1 mm of rainfall or more, the distribution of R has a pattern that is a major characteristic of hourly rainfall at the site. The distribution of R for a particular site is created by extracting all the values of R at the site for days where the rainfall was greater than or equal to 1 mm, for the entire length of record. The computed R values are then collated into 20 ranges, which were used by Boughton (2000) and are shown in Table 3.7. The distribution of R thus shows the proportion of all values of R in each of the ranges.

Table 3.7 Ranges used when collating R values

No.	Range	No.	Range	No.	Range	No.	Range
1	0.0417-0.075	6	0.275-0.325	11	0.525-0.575	16	0.775-0.825
2	0.075-0.125	7	0.325-0.375	12	0.575-0.625	17	0.825-0.875
3	0.125-0.175	8	0.375-0.425	13	0.625-0.675	18	0.875-0.925
4	0.175-0.225	9	0.425-0.475	14	0.675-0.725	19	0.925-0.975
5	0.225-0.275	10	0.475-0.525	15	0.725-0.775	20	0.975-1.000

The distributions of R for two sites in differing climates are shown in Figure 3.15. Jonkershoek (Station Jnk19a) in the Western Cape is located in a winter rainfall region, whereas Ntabamhlope (Station N23) in the KZN midlands is located in a summer rainfall region.

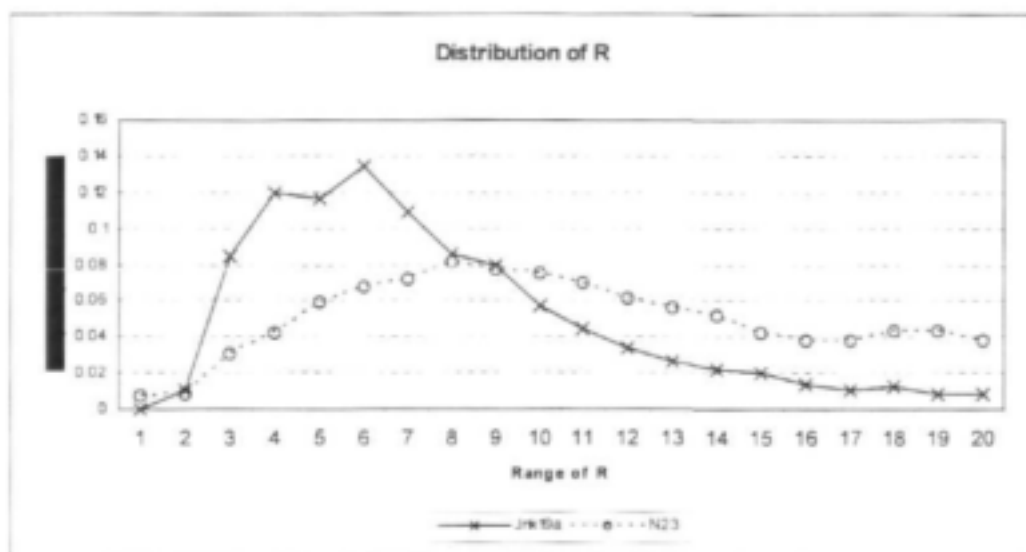


Figure 3.15 Frequency distributions of R at Stations Jnk19a (in the W. Cape) and N23 (in KZN)

From Figure 3.15 it is evident that the majority of the days at Jonkershoek fall into Range 6 in Table 3.7 and have many low values of R (mean $R = 0.385$) indicating that there is a tendency for more uniform rainfall. The distribution of R at Ntabamhlope (mean $R = 0.537$) shows a larger proportion of the days having higher values for R. This indicates that at Ntabamhlope larger fractions of the daily rain fall in a single hour, which is typical of the convective storms in the summer rainfall region.

3.3.1.3. Calculating the Other 23 Hourly Fractions

If $R = 1.0$ then all of the rainfall fell in a single hour and the other 23 hourly fractions must be 0. If $R = 0.04167$ then the other 23 hourly fractions must equal 0.04167. If, however, R is slightly less than 1.0 it is probable that the rest of the day's rainfall fell in 1 or 2 other hours, resulting in the remaining 21 or 22 hours having zero rainfall. Conversely, if R is slightly greater than 0.04167 then the other 23 values will probably be slightly less than, but close to, 0.04167. This is important to note as it indicates that the value of R has a strong influence in determining the other 23 hourly fractions of rainfall.

In order to determine the other 23 hourly fractions, the 24 hourly fractions for every day on record were ranked in order of magnitude, with R being the largest value on each day. This was done for 157 stations in South Africa. These ranked series, from all 157 sites, were then pooled together and the values for each rank were averaged in each of the 20 ranges of R shown in Table 3.7. This resulted in 20 averaged ranked series of hourly fractions, one for each range of R.

Once all 24 hourly averaged fractions have been determined for each range of R they can be used to create daily temporal patterns of rainfall. The following two sections contain a description of how these 24 hourly fractions are arranged to recreate possible realisations of the temporal distribution of daily rainfall.

3.3.1.4. Clustering of Hourly Rainfalls

In order to cluster the 24 hourly fractions, the data from all stations were again processed to calculate the highest 2-hour fraction of the daily total, the highest 3-hour fraction, the highest 6-hour fraction and the highest 12-hour fraction on each day. As for the ranked series, all of these fractions were then averaged within the range of R in which they occurred. This resulted in an average 2-hour fraction, 3-hour fraction, 6-hour fraction and 12-hour fraction of the daily total for each of the 20 ranges of R.

Using the abovementioned ranked sequences, a computer program was used to check the sum of the first value in the ranked series with each of the other 23 hourly fractions in order to find which of the 23 values gave the best match to the average 2-hour fraction for the respective range of R. After identifying the value in the series, which when added to the first value best approximated the average observed highest 2-hour fraction, the remaining 22 hourly values are scanned to find which value should accompany the 2-hour fraction to approximate the average highest 3-hour fraction. The remaining 21 values are then searched for a combination of 3 values to form the average highest 6-hour fraction, and then a search is performed for the next combination of 6 values to form the average highest 12-hour fraction. Performing this for each range of R resulted in 20 clustered sequences. The next step was to arrange these clustered sequences into temporal patterns.

3.3.1.5. Daily Temporal Patterns of Hourly Rainfalls

Schmidt and Schulze (1987) derived four design rainfall temporal distributions to be used for different regions in South Africa. This suggests that a single distribution can be used to represent the temporal distribution of rainfall for a particular region. This, however, is not realistic and analysis of the rainfall data shows that at a given location there are several temporal patterns ranging from nearly uniform rainfall to highly variable rainfall. Furthermore, the peak intensity can occur during any hour of the day, adding to the variability of temporal rainfall patterns. In order to account for the variability of temporal patterns of rainfall, several temporal distributions should be employed.

The hour of day when the highest intensity rainfall occurred was determined for each station. The results show a definitive distribution for the timing of peak rainfall occurrence at a particular location. As shown in Figure 3.16 for Station Jnk19a, the hour of maximum rainfall has a somewhat uniform distribution, indicating that the hour of maximum rainfall has a reasonably equal probability of occurring in any hour on a particular day. Station N23, however, has a sinusoidal-like distribution with the majority of days having the peak rainfall falling during the late afternoon and evening.

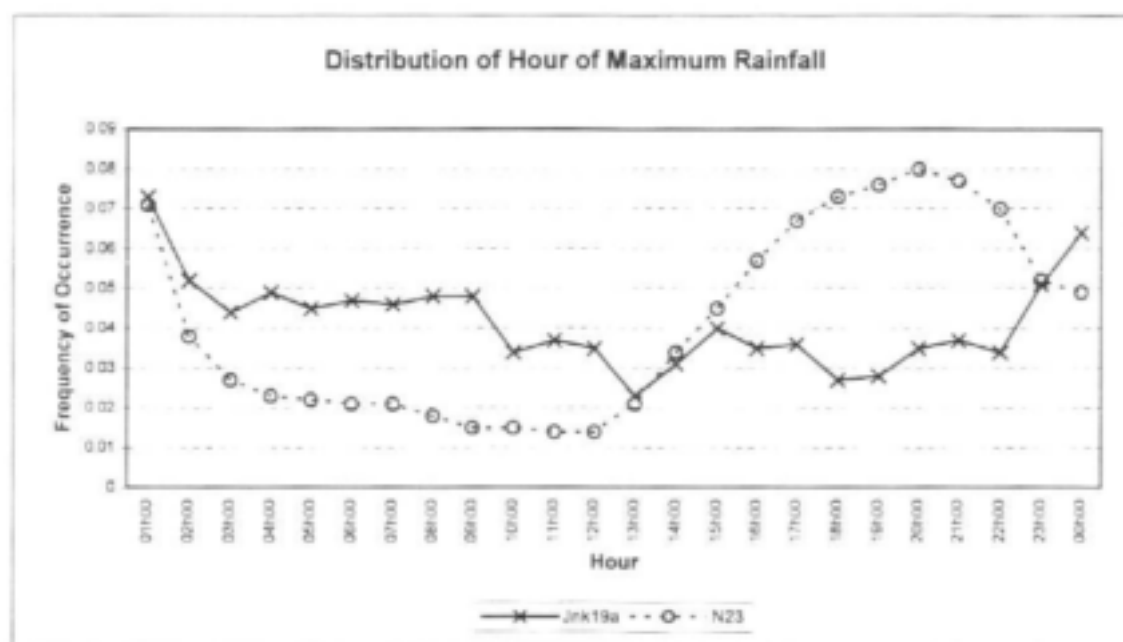


Figure 3.16 Frequency distributions of the hour of maximum rainfall at Jonkershoek (in the W. Cape) and Ntabamhlope (in KZN)

When applying the disaggregation method, a random number is used to select the hour of maximum rainfall from the distribution of the hour of maximum rain for the site of interest. This differs from the work done by Boughton (2000) as in that study no distinct distribution was found for the time of maximum rainfall and hence the hour of maximum rainfall was selected at random.

Using the clustered sequences established above, and then accounting for all permutations when the hour of maximum rainfall can occur, 24 arrangements of the clustered sequences can be created. The combination of these 24 arrangements with the 20 possible ranges of R results in 480 different temporal patterns, as opposed to one averaged distribution currently commonly used in South Africa. These range from uniform to non-uniform with the possibility of the hour of maximum rainfall occurring in any hour of the day. Figure 3.17 contains a sample of the different temporal distributions that the disaggregation model can produce.

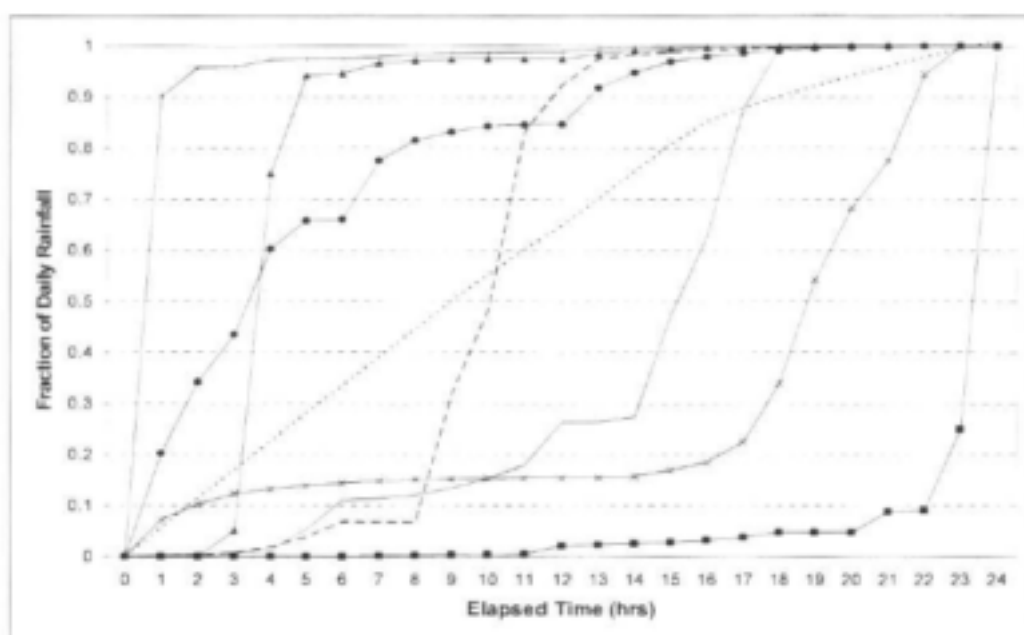


Figure 3.17 Samples of the different temporal distributions generated by the disaggregation model

3.3.2. Model Testing and Results

In order to quantify the simulated performance of the disaggregation model, a similar approach to that used by Smithers and Schulze (2000b) was employed. Moments and other event characteristics computed from the disaggregated rainfall series are compared in Section 3.3.2.1 to the equivalent values computed from the observed data. Similarly, design rainfall depths computed from the disaggregated rainfall series are compared to the equivalent values computed from the observed data in Section 3.3.2.2. Both measures of quantifying the model performance were carried out at 15 test stations, details of which are given in Knoesen (2005). The data from all 15 test stations were excluded from model development.

3.3.2.1. Moments and Statistics

The two random processes that occur within the disaggregation model, viz., the selection of the value of R and the timing of the hour of maximum rainfall, introduce stochastic variability in the output. At each of the selected test stations the stochastic variability was simulated by generating one hundred disaggregated series. A frequency analysis was performed on the 100 sets of disaggregated values for all statistics and durations considered. High-Low bar graphs depicting the observed moments and the 25th and 75th non-exceedance percentiles of the 100 sets of disaggregated values are used to graphically depict the performance of the model. The poorest results were achieved at Station 0435019, located at Ottosdal, while the best results were achieved at Station Sacfs, located at Umhlanga. The results from disaggregating the 24-hour data at Stations 0435019 and Sacfs are shown in Figure 3.18 and Figure 3.19 respectively.

It can be seen in Figure 3.18 and Figure 3.19 that the disaggregation model performs equally well in simulating the mean, standard deviation and skewness at Stations 0435019 and Sacfs for the range of durations considered. It was expected that the mean rainfall for all levels of aggregation should be simulated extremely well as a consequence of the method of disaggregation. The model tends to be less capable of simulating certain event characteristics and statistics such as event duration and dry probability. This is a weakness in the disaggregation model and suggests that more work needs to be done on refining the sequencing of the hourly rainfalls. The distinguishing factors between the best and worst simulations are the lag 1 – 10 autocorrelations, and is shown by Knoesen (2005) to be directly related to the quality of the data at the respective sites.

Furthermore, it is evident from Figure 3.18 and Figure 3.19 that the dry probabilities, and hence the event durations, are better simulated at Station 0435019 than at Station Sacfs. It is postulated that this may be attributed to the distribution of R at Station 0435019, which has a mean value of R (R_{mean}) of 0.607, whereas Station Sacfs has an R_{mean} of 0.499. This appears to be a shortcoming of using a single distribution of R to represent all rainfall depths. Although the disaggregated data will have the correct overall distribution of R , it is likely that the tails of the rainfall distribution will not receive the correct values of R .

3.3.2.2. Extreme Rainfall Events

Similar to the procedures used by Smithers and Schulze (2000b), design rainfall depths were calculated using the General Extreme Value (GEV) distribution fitted to the Annual Maximum Series (AMS) by L-moments, for the observed data and for each of the 100 disaggregated series generated from the disaggregation model. Design values for the 2, 5, 10, 20, 50, and 100-year return periods were computed for durations of 1, 2, 3, 4, 6, 8, 10, 12, 16, 20 and 24-hour. For each duration and return period, a frequency analysis was performed on the 100 values computed from the disaggregated rainfall series generated by the disaggregation model. High-Low bar graphs depicting the observed design rainfall computed from the observed data and the 25th and 75th non-exceedance percentiles of the design rainfall computed from the 100 disaggregated rainfall series were used to evaluate the performance of the model. Examples of model performance, with respect to design rainfall estimation, are shown in Figure 3.20, which depict the worst (Station 0028748 at George) and best (Station 0474680 at Carletonville) simulations.

The poor performance observed when estimating design floods at Station 0028748 seems to be related to the distribution R , i.e. the station that displayed the best results has an $R_{\text{mean}} = 0.60$, whereas the station with the worst results, Station 0028748, has an R_{mean} value = 0.45. After analysing all the test stations it was found that the stations with the highest R_{mean} values gave the best results. This is because on those days when smaller rainfall events (± 1 mm) occurred it is likely that the all the day's rainfall fell within a few hours, thus unduly influencing the distribution R . Although this will affect all the stations used, it appears that the error is exacerbated for those stations with lower mean R values. It is postulated that the use of different distributions of R to represent rainfalls of differing magnitudes will improve the performance of the rainfall disaggregation model, particularly in the estimation of design rainfall.

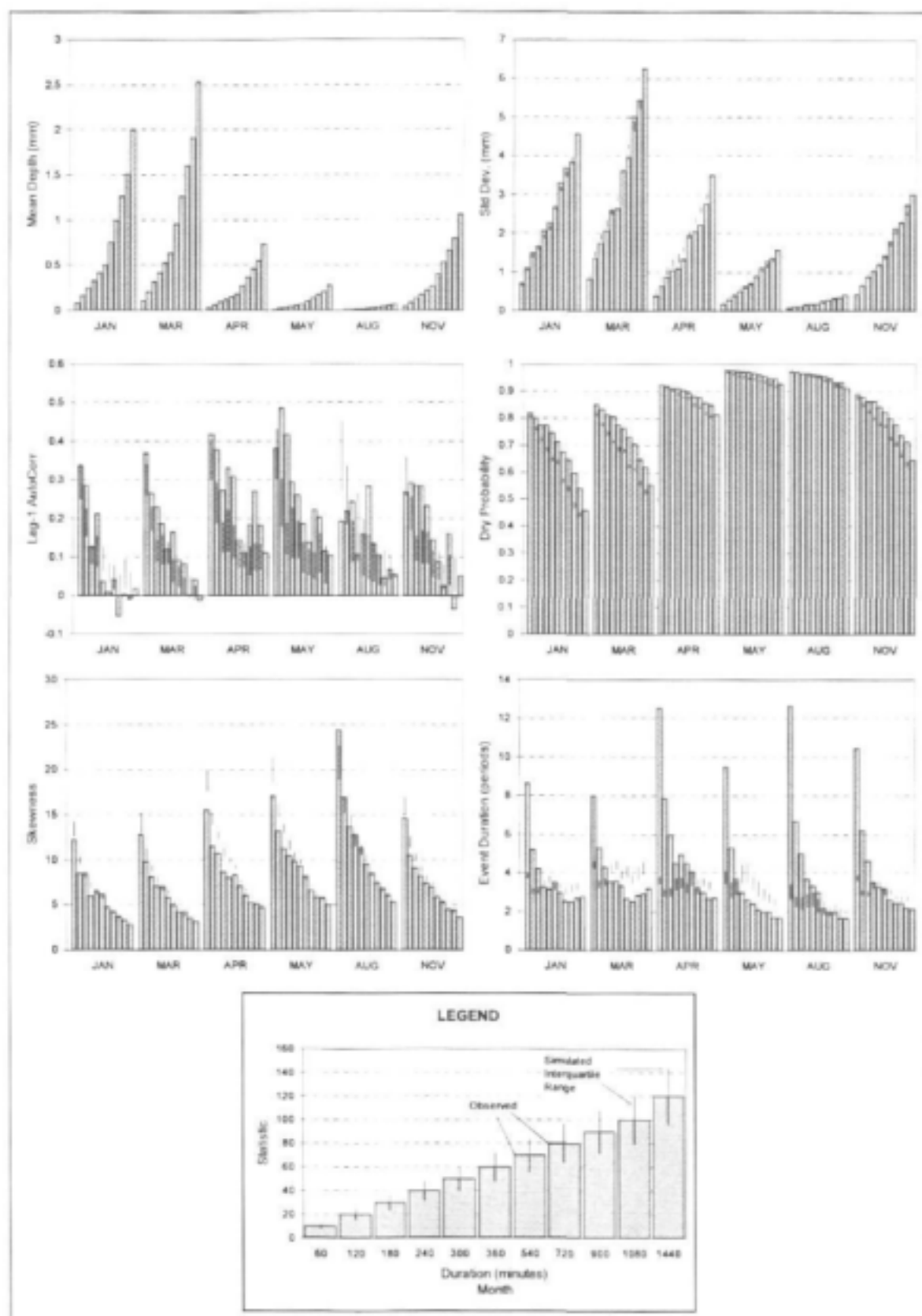


Figure 3.18 Simulated performance of the disaggregation model at Station 0435019

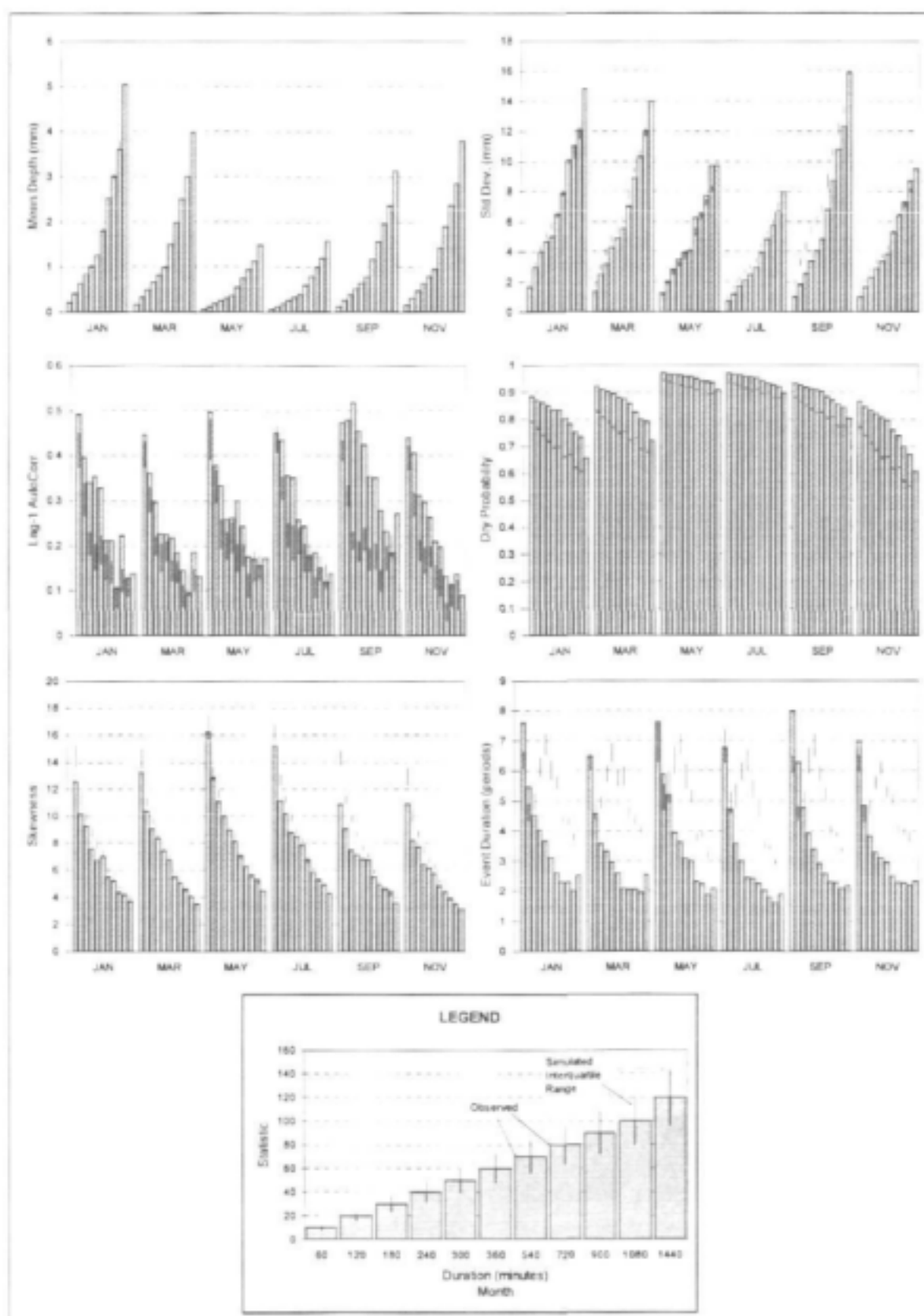


Figure 3.19 Simulated performance of the disaggregation model at Station Sacfs

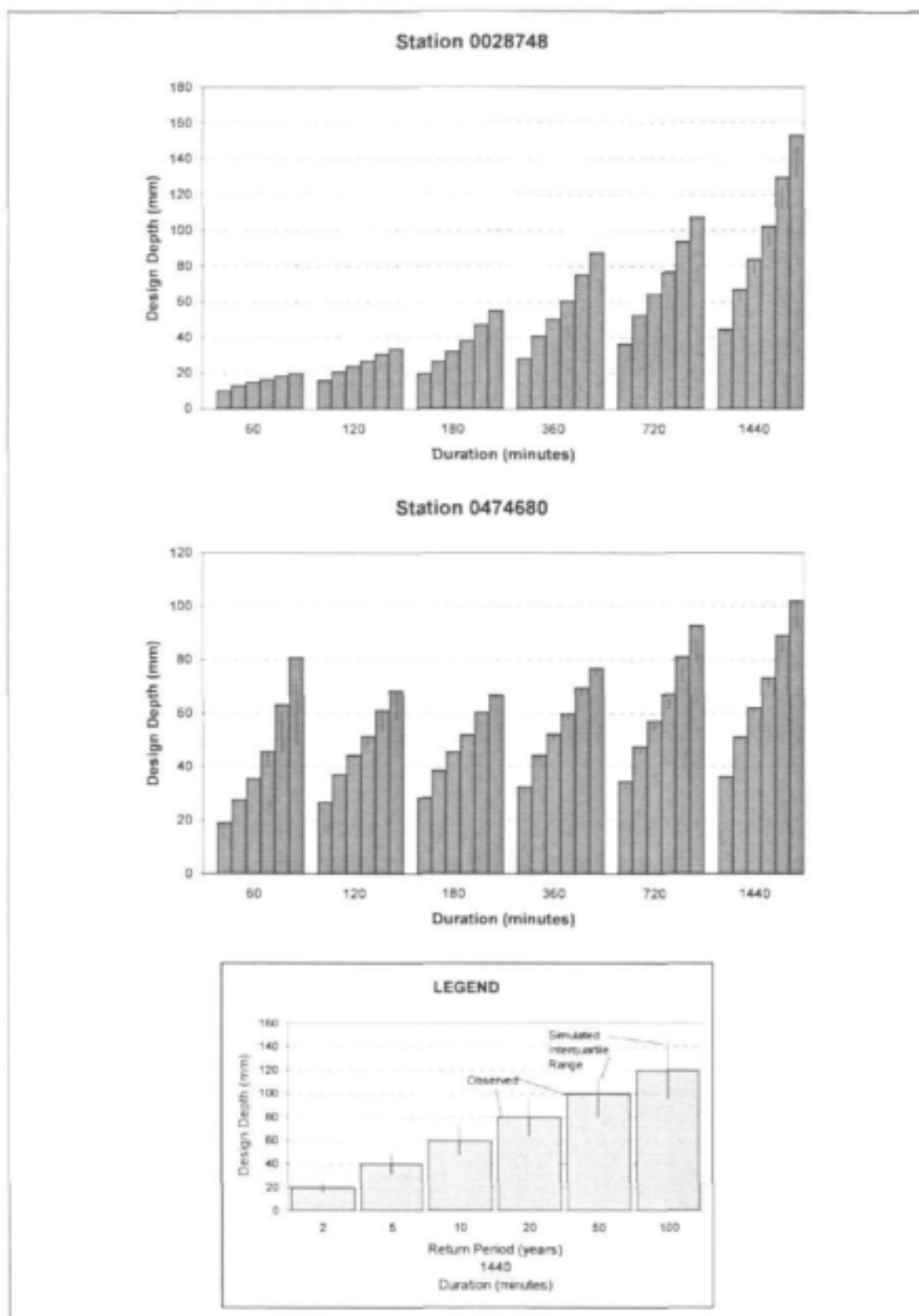


Figure 3.20 Design rainfall estimated using disaggregated data for Stations 0028748 (George) and 0474680 (Carletonville)

3.3.2.3. Regionalising the Methodology

In order to apply the methodology at sites where no short-duration data are available, it is necessary to regionalise the methodology. Assessing the model performance using the moments and statistics of the disaggregated series, as well as the design rainfall computed from the disaggregated series, results obtained by Knoesen (2005) show that applying the model using regionalised input gives very similar results to those obtained when at-site short duration data are available. For details regarding the approach used to regionalise the methodology, the reader is referred to Knoesen (2005).

3.3.3. Discussion

The rainfall disaggregation model developed by Boughton (2000) was intended to be used only for design flood purposes, and hence only focused on disaggregating larger daily rainfalls (>15 mm). In order to achieve the objectives of this study, the methodology was modified. All days for which the aggregated 24-hour rainfall total was greater than or equal to 1 mm were used to develop the disaggregation model, which is used to disaggregate all non-zero 24-hour rainfalls.

Further modifications were made to the methodology regarding the distribution of the time when the hour of maximum rainfall occurred. It was evident that the rainfall data at different stations displayed different distributions for the hour of maximum rainfall. The distribution of the hour of maximum rainfall at each station was computed, and random sampling from the respective distributions is performed. This differs from the original Boughton (2000b) model where the hour of maximum rainfall was determined by random sampling from a uniform distribution.

The resulting model is capable of producing 480 different temporal patterns with ranging levels of uniformity. The distribution of R and the distribution of the hour of maximum rainfall for each station determine which of the 480 possible temporal patterns to select for a particular day's rainfall.

Two measures were employed in order to quantify the performance of the disaggregation model. Firstly, moments and other event characteristics were computed from the disaggregated data and compared to the equivalent values computed from the observed data. Secondly, design rainfall depths were computed from the disaggregated data and compared to the equivalent values computed from the observed data.

Owing to the stochastic nature of the disaggregation models, 100 disaggregated series were generated for each independent test location and frequency analyses were performed. High-Low bar graphs depicting the observed moments and the 25th and 75th non-exceedance percentiles of the 100 sets of disaggregated data were used to graphically depict the adequacy of the model.

The results from the model where at-site short duration data are available indicate that the model is able to produce synthetic hourly rainfall series which resemble the general distribution of the observed hourly data for a particular site. However, the results also indicate that the model is less capable of simulating some of the statistics considered i.e. the probability of dry periods and design rainfalls for selected return periods.

The relatively poor results obtained for the various lag autocorrelations from the disaggregated rainfall is postulated to be the result of the way that the hourly rainfalls

are sequenced. Although the lag autocorrelations were better simulated for longer and better quality data, it is suggested that a different method of sequencing the disaggregated hourly rainfalls may improve the model in this respect.

The methodology was regionalised, which is a further modification to the methodology employed by Boughton (2000), and the results obtained when using the regionalised input to the model are similar to those obtained when at-site information is available, for both statistical and extreme value measures of model performance. This is a positive result as it implies that the model can be used to reasonably disaggregate daily rainfall at locations in South Africa where no short duration data are available.

3.3.4. Conclusions

From this component of the study it can be concluded that:

- The temporal distribution of rainfall is an important factor affecting the timing and magnitude of peak discharge from a catchment.
- A methodology for the disaggregation of daily rainfall to produce hourly increments which aggregate to equal the observed daily values has been identified and applied in South Africa.
- The methodology is shown to reproduce the general distribution of rainfall relatively well, both when observed short duration data are available as well as in the absence of such information.
- Refinements need to be made to the methodology in order to improve sequencing and the correlation structure of the disaggregated rainfall.

3.3.5. Recommendations for Future Research

It is postulated that the weakness of the model in simulating both extremes of the rainfall spectrum (dry probabilities and design rainfall) is a result of the use of a single distribution of R to represent the entire range of magnitudes within a rainfall record. It is recommended that, for a particular rainfall record, the data be collated according to the daily rainfall total, using pre-determined ranges, and a distribution of R be calculated for each of these ranges. It is further recommended that more research be undertaken on how to sequence the disaggregated hourly rainfalls, in order to improve the simulation of the structure of the rainfall, as measured by the lag autocorrelations, number of events and event durations.

It is further recommended that the disaggregation model be linked with a reliable daily rainfall generator. This would facilitate the generation of long sequences of hourly data for any location in South Africa which could be used for modelling of water resources and design flood estimation.

3.4. Spatial Distribution of Rainfall

In the application of information derived from rainfall data in the fields of hydrology, engineering and agriculture, it is becoming increasingly important to know, or at least have a reasonable estimate of rainfall in space as well as time, and in more detail than it is possible to deduce from the data collected at raingauges in a sparse network (Pegram and Seed, 1998). Traditionally, mathematical interpolation techniques have been used to interpolate rainfall data from a raingauge network to estimate the rainfall at ungauged sites. However, a rainfield estimated using mathematical interpolation does not accurately represent the "true rainfall" field. Therefore, other techniques

have been developed to improve the estimation of the spatial distribution of rainfall. Some of these methods generate synthetic rainfall values using statistical models (e.g. Pegram and Seed, 1998; Pegram and Clothiers, 2001), or are models based on the physical properties of a rain cell or cloud (Gupta and Waymire, 1990), or are techniques that derive a sound link between the radar field and the raingauge data (Todini, 2001; Ehret, 2002). Merged rainfall fields, derived from radar and raingauges, are currently the best estimate of the "true" rainfall field.

Rainfall is highly variable in space and time and the reliable estimation of design floods from rainfall using continuous simulation modelling (CSM) and other hydrological studies require detailed information of the rainfall distribution in both space and time and for a long period of record. Raingauges measure rainfall directly and rainfall depth accumulated over the period of interest is measured with a high degree of accuracy at points where the gauges are located. However, raingauge networks are too sparse to capture the spatial variability of rainfall (Wilson and Brandes, 1979). Radar, on the other hand, measures a volume-averaged returned signal power which is converted into rainfall in two steps: first to a reflectivity factor (Z), and then to rainfall units (R). Although indirect, radar estimates of rainfall are continuous in space and provide information on the spatial variability of rainfall. However, rainfall estimated using radar lacks accuracy at a point which raingauges provide (Wilson and Brandes, 1979).

Merging of radar and raingauge data using a merging technique enables the best estimate of the spatial distribution of rainfall to be made. However, the length of available radar records in South Africa is currently limited and most hydrological models require a long input sequence of rainfall records. Therefore, the objectives of this component of the study reported in this section are to validate and verify the merging technique developed by Sinclair (2004) and to develop relationships between average daily rainfall depths for a sub-catchment, estimated by the merging of radar and raingauge data, and rainfall measured by daily raingauges, which are selected to represent the rainfall in the sub-catchment. These relationships which are derived using the relatively short period of available radar data can be used with the long records of raingauge data to improve the estimation of rainfall falling over a catchment. This study was performed using information from the Liebenbergsvlei catchment where the required radar and raingauge data were available.

In addition to using observed rainfall data as input to a CSM, long sequences of runoff may be simulated by a CSM to estimate design floods by using a stochastic space-time rainfall model. Hence, the String of Beads Model (SBM) developed by Pegram and Clothier (2002) was used to generate synthetic rainfall series for the Liebenbergsvlei catchments. The SBM model is able to produce rainfall values at a spatial resolution of 1x1 km with a 5 minute temporal resolution. The SBM is a high-resolution space-time model of radar rainfall images, which takes advantage of the detailed spatial and temporal information captured by weather radar and combines it with the long-term seasonal variation captured by a network of daily raingauges. Statistics from a 50 year period of generated rainfall values were compared with the statistics computed from a 50 year raingauge data series, and it was found that the generated rainfall values mimic the rainfall data from the raingauges reasonably well.

The objectives of this component of this project were to:

- Develop or adopt and assess a methodology to improve the estimation of the average depth and spatial distribution of rainfall over a catchment using radar images and point rainfall data.
- Assess the influence of the improved catchment rainfall on streamflow simulated by a CSM.
- Evaluate a detailed space-time stochastic rainfall model to generate long sequences of rainfall over a catchment for use in a CSM.

Full details and results of the study reported in this section may be found in Frezghi (2005).

3.4.1. Study Area

The Liebenbergsvlei catchment is a sub-catchment of the Vaal River Catchment and is located near Bethlehem in the Free State Province of South Africa, as shown in Figure 3.21. The Liebenbergsvlei Catchment is in a relatively dry region of South Africa and has an area of 4694 km² which receives an average annual rainfall total of 650 mm (Pegram and Sinclair, 2002). Most of this precipitation falls during the summer season, which ranges from October to February. The mean annual runoff depth from the catchment for the 21 year period from 1978 up to and including 1998 was 38 mm (Midgley *et al.*, 1994b). Rainfall has been intensively monitored in the Liebenbergsvlei catchment both by raingauges (daily and recording tipping buckets gauges) and by radar. Therefore, the Liebenbergsvlei catchment was selected as a test site for this study.

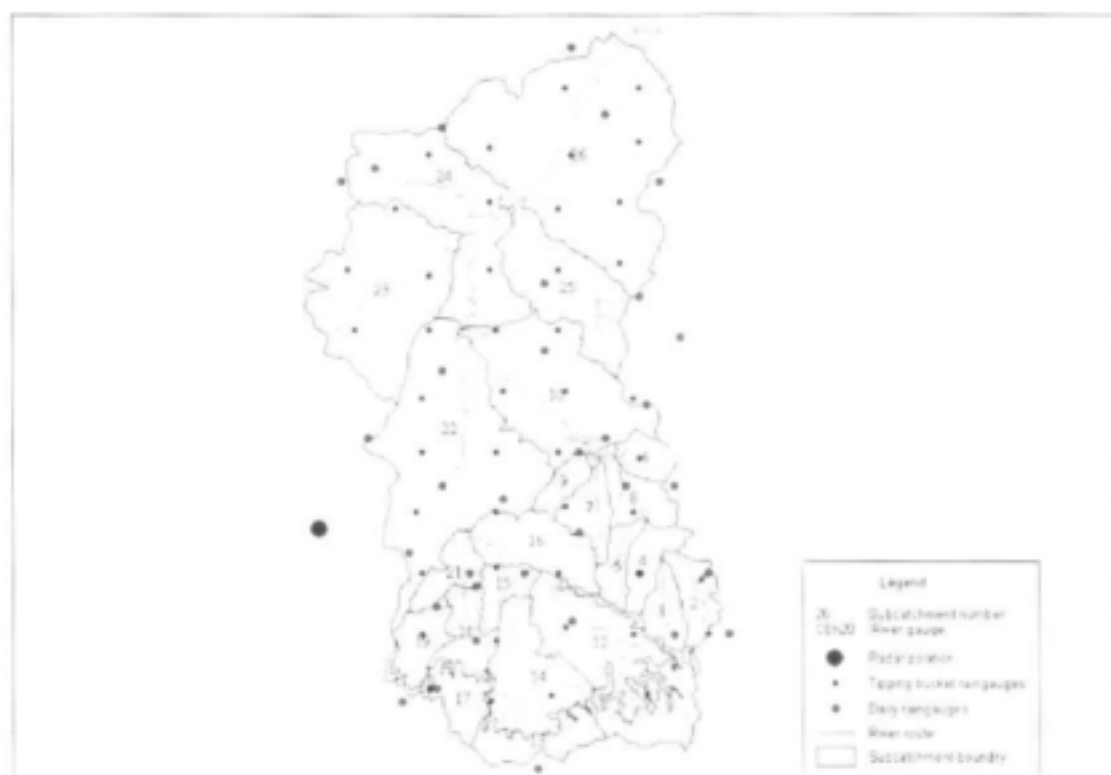


Figure 3.21 Location of raingauges and gauging weirs in the Liebenbergsvlei Catchment

3.4.2. Conditional Merging

The merging of rainfall values from raingauges and radar exploit the complementary characteristics of the two rainfall measuring techniques. Several methods have been developed for merging raingauge and radar data and the merged values have generally produced good results in terms of bias reduction, although little attention has been given to the reduction of variance (Todini, 2001). The different nature of the errors, which implies their independence (Seo *et al.*, 1990), can be exploited to produce unbiased and more reliable estimates of rainfall. Following this idea, Todini (2001) proposed a Bayesian combination technique, based on the use of block Kriging and a Kalman filter, which seeks to eliminate the bias in meteorological radar estimates of precipitation and to produce precipitation estimates which have a minimum variance on pixels of variable sizes.

Radar produces an image of the unknown true rainfall field which is subject to several well-known sources of error, for example as detailed by Wilson & Brandes (1979) and Habib and Krajewski (2003), but retains the general covariance structure of the true precipitation field. The information from the radar can be conditioned using the spatially limited information obtained by interpolating between raingauges to produce an estimate of the rainfall field that contains the correct spatial structure, while being constrained to the raingauge data. This process is illustrated in Figure 3.22. The conditional merging technique of Ehret (2002) makes use of ordinary kriging to derive information from the observed gauged rainfall data.

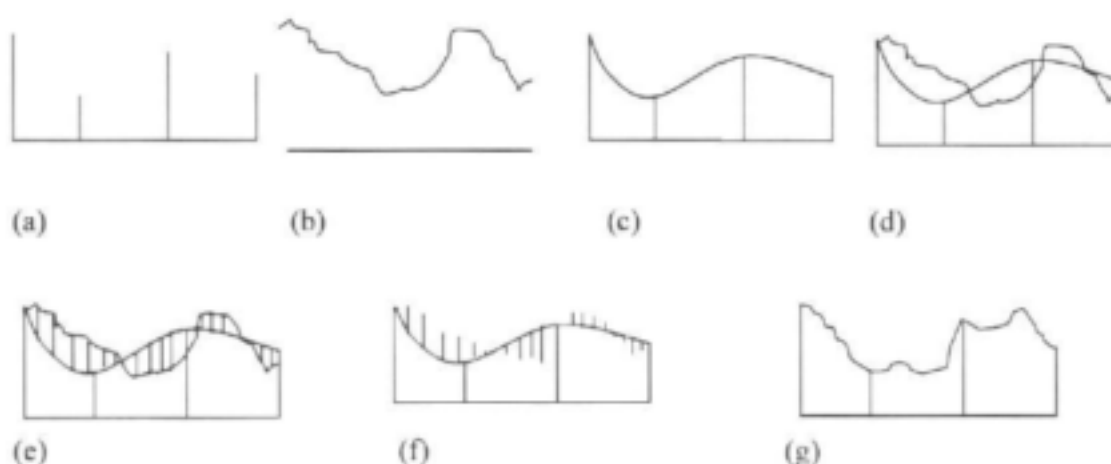


Figure 3.22 Conditional rainfall merging process (Pegram and Sinclair, 2004)

With reference to Figure 3.22, the conditional merging process described by Pegram and Sinclair (2004), and utilised in this study, is as follows:

- The rainfall field is observed at discrete points by raingauges.
- The rainfall field is also observed by radar on a regular, volume-integrated grid.
- Kriging of the raingauge observations is used to obtain the best linear unbiased estimate of rainfall on the radar grid.
- The radar pixel values at the raingauge locations are interpolated onto the radar grid using Kriging.

- At each grid point, the deviation between the observed and interpolated radar value is computed.
- The field of deviations obtained from (e) is applied to the interpolated rainfall field obtained from Kriging the raingauge observations.
- A rainfall field that follows the mean field of the interpolated raingauge data, while preserving the mean field deviations and the spatial structure of the radar field, is obtained.

3.4.3. Relationship between Daily and Merged Rainfall Fields

Many hydrological studies require a long temporal sequence of spatially detailed and accurate rainfall information. However, the available rainfall data in South Africa are either long series of point rainfall, with no spatial information (raingauge data), or detailed spatial and temporal rainfall information, but with a limited period of record (radar and satellite rainfall values). Therefore, a relationship between the best estimate of the average daily rainfall depth in a sub-catchment, obtained from merging the raingauge data with rainfall derived from the short period of available radar data, and daily rainfall data from raingauges, is developed in this section. The derived relationship can then be used to adjust the long period of daily rainfall data to better represent the rainfall in the catchment.

The merging procedure was assessed in two stages. Firstly, the merging process developed by Sinclair (2004), which is based on the conditional merging technique proposed by Ehret (2002), was validated using rainfall data from the tipping bucket raingauges used in the conditioning of the radar rainfall values and, secondly, the merging procedure was verified using daily rainfall data from raingauges which were not used in the calibration of the merging process. Thereafter, the reliability of the relationship between the averaged merged rainfall values for the sub-catchments, obtained by combining the radar and raingauge data, and the raingauge data selected to represent the areal rainfall of the sub-catchment, is investigated.

3.4.3.1. Validation of the Merging Process

In the merging process, spatial interpolation of point measurements utilise Kriging of the raingauge observations to obtain the best unbiased estimate of rainfall at the radar pixels. The observed rainfall data, for the pixels where the raingauges are located, are fixed in the merged rainfall images without adjustment. Therefore, the average merged rainfall values at the same location as the conditioning raingauges should be equal to the measured rainfall at the conditioning gauge. At the pixels where the conditioning raingauges are located, the merging algorithm has "exact" knowledge of the measured rainfall, and a 1:1 linear relationship (best fit straight line, $Y = x$; $R^2 = 1$) between the average merged and conditioning gauge values was expected. Anywhere else in the merged field it was expected that some error would be present between the true rainfall field and the merged estimate ($Y = ax + b$; $R^2 < 1$), for example at locations of raingauges not used in the merging process. However, the closer a raingauge is to the location of a raingauge used in the merging procedure, the more accurate the merged value was expected to be. In this study the tipping bucket raingauges in the Liebenbergsvlei catchments were used as conditioning raingauges.

The validation was performed using all tipping bucket raingauges used in the merging procedure and all days where radar images were available to this study (2 October 1998 to 31 March 1999) are included in Figure 3.23, where a linear regression of

$y = 0.937x - 0.2326$ ($R^2 = 0.9225$) was obtained. According to the merging process developed by Sinclair (2004), radar pixels with no rain are masked (i.e. excluded) and hence in regions where the radar registered no rain, the merged value is assigned zero rainfall, even though a raingauge in the region may have registered rainfall. This is a trade-off between being wrong at the raingauge in a few cases and having rainfall over the entire data domain which means many "false rainfalls" elsewhere in the region of interest (Sinclair, 2004). When the merged values with masked zero values, as a consequence of no rain registered by the radar are removed from the data, a near perfect regression relationship of $y = 1.001x - 0.036$ ($R^2 = 0.9996$) was obtained, as shown in Figure 3.24. From the validation of the merged rainfall with the tipping bucket raingauges used in the conditioning of the radar images, it is evident that the merging procedure developed by Pegram and Sinclair (2004) successfully assigns rainfall values to the merged pixels at the locations of the raingauge used in the conditioning of the radar images.

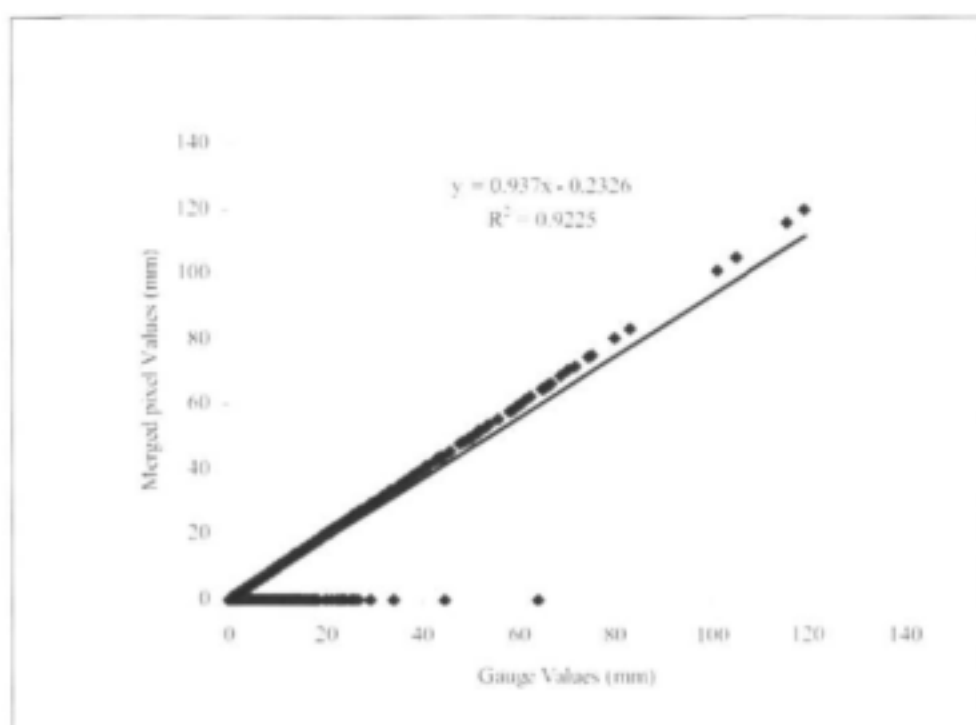


Figure 3.23 Validation of the merging process for all tipping bucket raingauges data

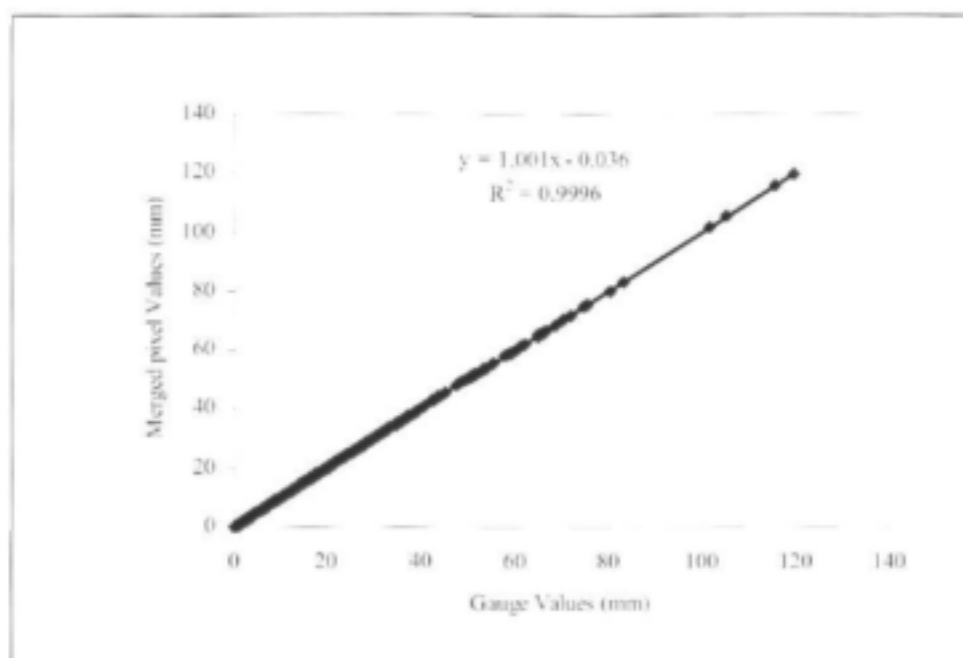


Figure 3.24 Validation of the merging process for all tipping bucket raingauge data after merged zero rainfall values resulting from no rainfall in the radar images were removed

3.4.3.2. Verification of the Merging Process at Raingauges Not Used in the Conditioning of Radar Images

Average merged pixel values at the location of daily raingauges, which were not used in the conditioning of the radar pixel, were compared to the gauged values. Raingauge 0331607W, located in Sub-catchment 22 as shown in Figure 3.25, is used as an example of the verification, where a relationship of $y = 1.0385x - 0.0675$ ($R^2 = 0.7018$) was obtained from the comparison of rainfall from the daily raingauge and the merged pixel values at the raingauge location, as shown in Figure 3.26. However, as explained above, some of the merged pixels are assigned zero values at pixels where the radar registered no rainfall and when these pixel values are removed from the analysis, the relationship improves to $y = 1.1814x + 0.1224$ ($R^2 = 0.8205$), as shown in Figure 3.27. Figure 3.28 shows the relationship between tipping bucket Raingauge L015, used in the conditioning of the radar rainfall images, and the average merged pixel rainfall values at the location of Raingauge 0331607W. The relationship between the point rainfall from the daily raingauge which was not used in conditioning of the radar rainfall images, and the average merged rainfall values of a pixel at the location of the daily raingauge, is a function of the radar rainfall values, the rainfall from the conditioning tipping bucket raingauges which are close to the daily raingauges, and the distance between the tipping bucket raingauges and daily raingauge. Therefore, Figure 3.28 also shows the characteristic rainfall pattern between the point rainfall from the tipping bucket raingauge and average merged rainfall values and it demonstrates the influence of the tipping bucket Raingauge L015 on the relationship between the point rainfall from daily raingauges and averaged merged rainfall values of pixels at the location of the daily raingauges.

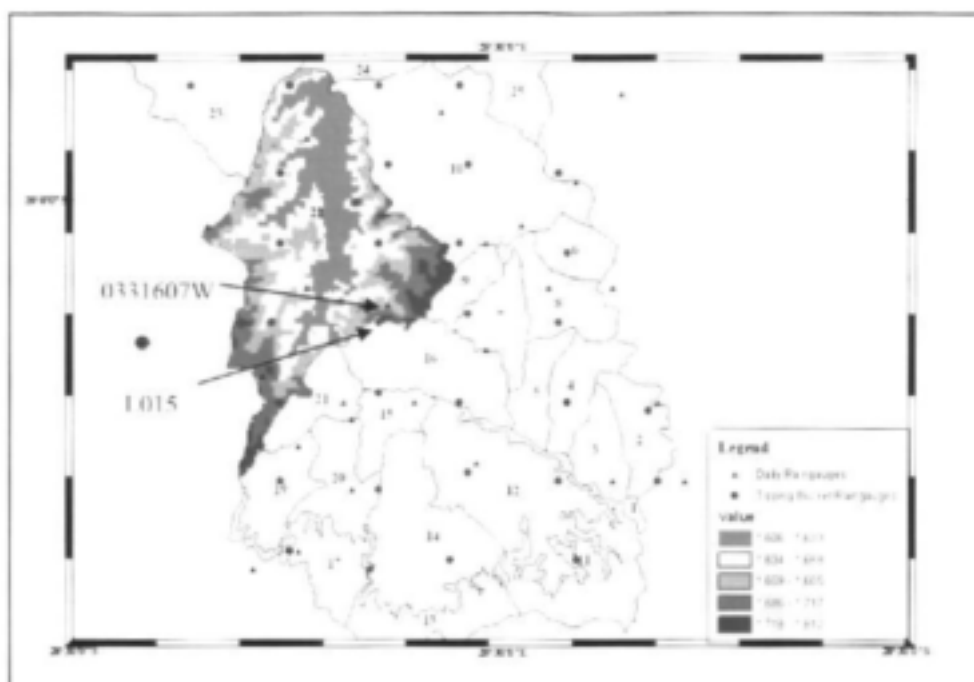


Figure 3.25 Location of Rain gauge 0331607W and altitude distribution in Sub-catchment 22

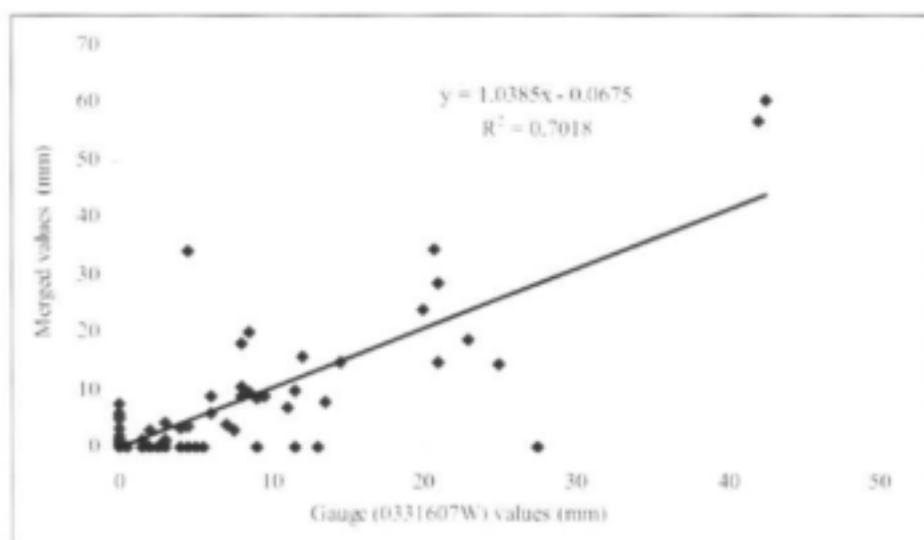


Figure 3.26 Comparison of daily rainfall from Rain gauge 03312607W, which was not used in the conditioning of radar rainfall values, and merged pixel rainfall values at the rain gauge location

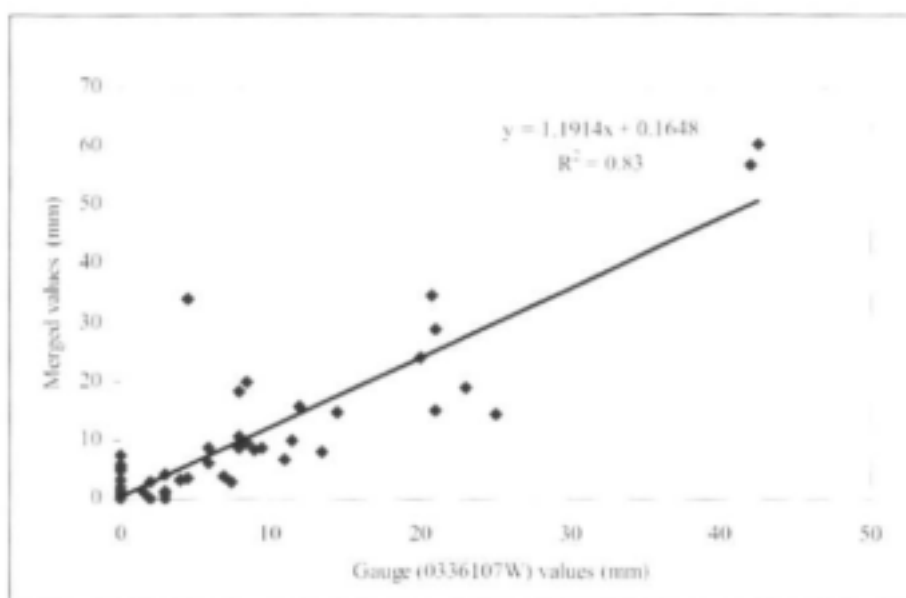


Figure 3.27 Comparison of daily rainfall data from Raingauge 03312607W, which was not used in the conditioning of radar rainfall values, and merged pixel rainfall values at the raingauge location after merged zero rainfall values resulting from no rainfall in the radar images were removed

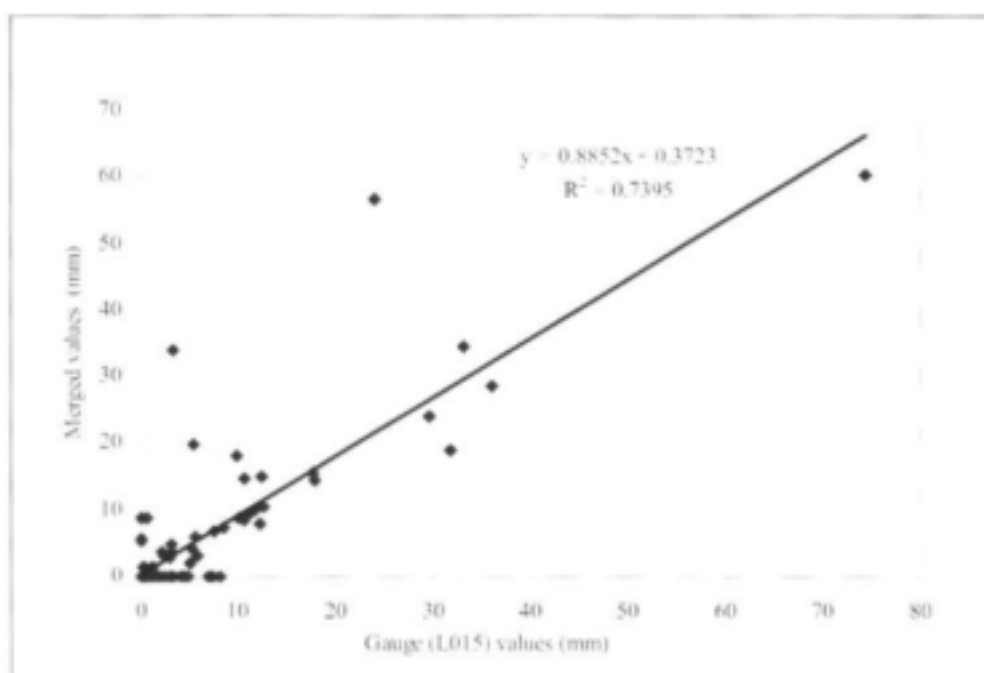


Figure 3.28 Comparison between merged pixel rainfall at location of daily Raingauge 03312607W and a nearby tipping bucket raingauge (L015)

3.4.4. Estimation of Sub-catchment Rainfall from Daily Rainfall

Relationships between the mean merged areal rainfall in the sub-catchments of the Liebenbergsvlei Catchment, and daily and tipping bucket raingauges selected to represent rainfall in the sub-catchments, were developed using data for the period when the merged rainfall values were available. These relationships can then be used to improve the estimation of catchment rainfall using historical data from the selected raingauges.

The downstream part of the Liebenbergsvlei catchment is relatively flat compared to the upstream part of the catchment. As a result, the size of the sub-catchments delineated by Jewitt *et al.* (1997), which were used in this study, are bigger in the downstream portion than in the upstream part. As shown in Figure 3.21, Sub-catchment 26 is situated in the lower portion of the study area and it is a relatively flat catchment with an area of 827.34 km². The location of Raingauge 0367601W is shown in Figure 3.29. The relationship between the average merged rainfall depth for Sub-catchment 26, obtained by averaging all the merged rainfall values at each pixel within the sub-catchment, and the daily raingauge (0367601W) values is shown in Figure 3.30. The linear relationship obtained ($y = 0.8157x$; $R^2 = 0.7275$) indicates that the daily raingauge generally overestimates the areal rainfall for the sub-catchment. Similar results were obtained for the other catchments.

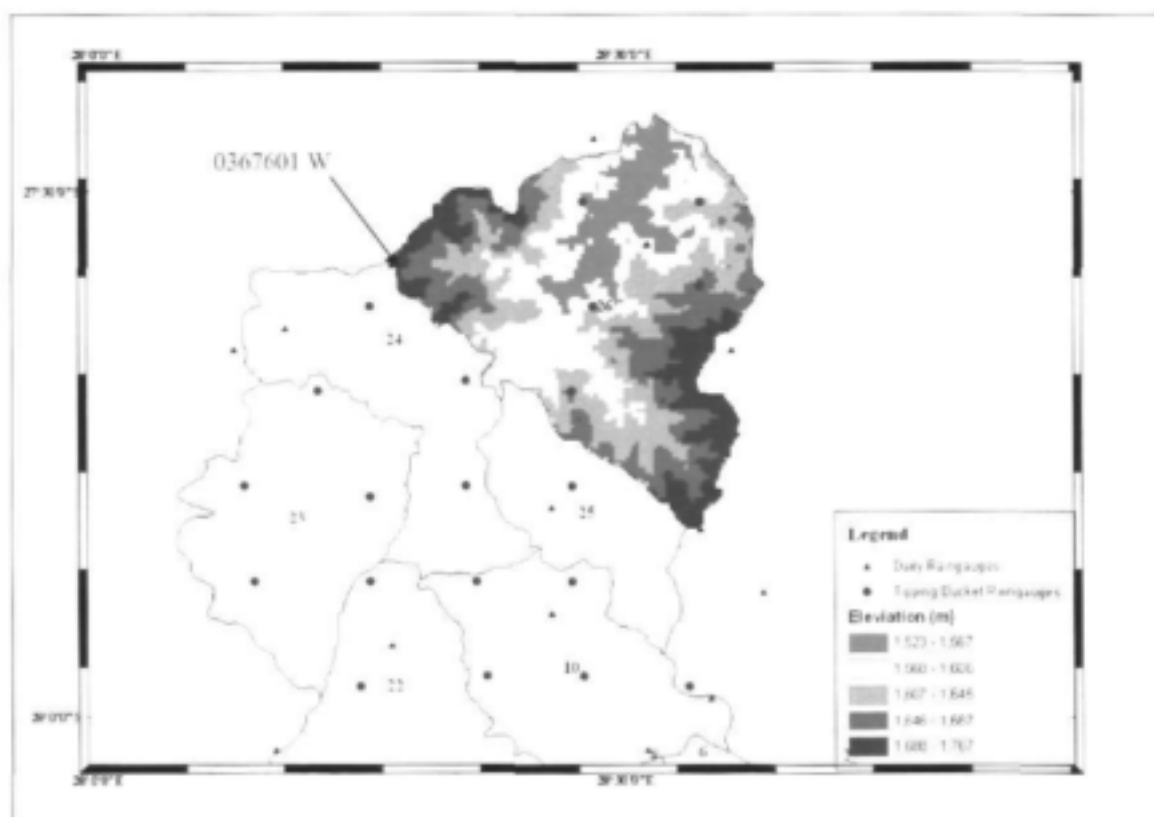


Figure 3.29 Location of Raingauge 0367601W and altitude map of Sub-catchment 26

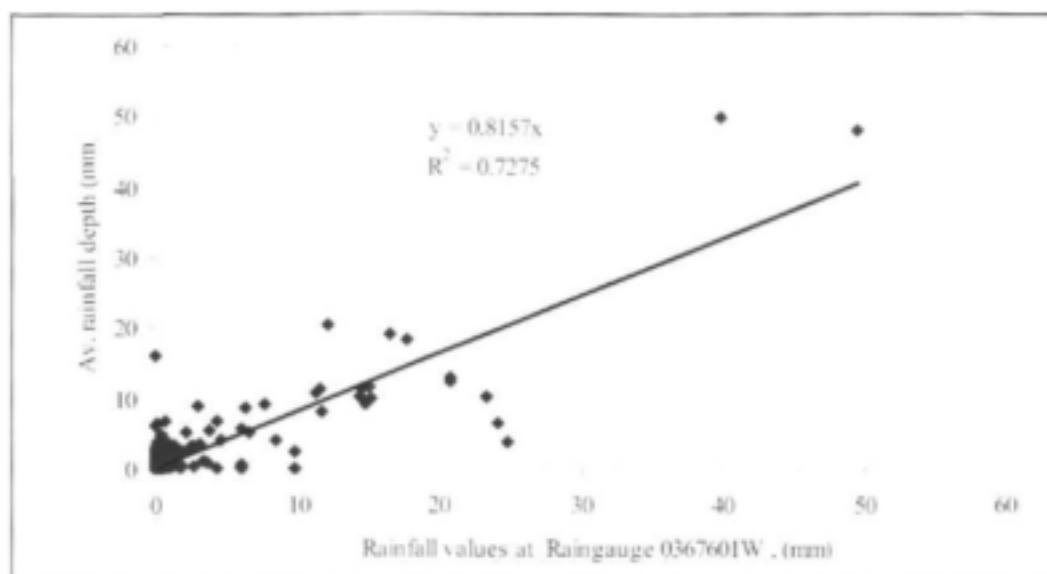


Figure 3.30 Relationship between average sub-catchment rainfall, derived from the merged rainfall field, and rainfall from Raingauge 0367601W in Sub-catchment 26

The spatial distribution of rainfall within Sub-catchment 26, computed from the merged rainfall surface, is relatively uniform for the period shown in Figure 3.31 where the standard deviation of rainfall on each day over the sub-catchment is shown, and where the majority of days have a standard deviation of less than 10 mm. The spatial uniformity of the rainfall over the sub-catchment implies that there is little or no orographic effect on the spatial distribution of the rainfall. Although the area of Sub-catchment 26 is relatively large, the spatial rainfall distribution of the sub-catchment is relatively uniform as shown in Figure 3.31. Therefore, a possible reason that rainfall at Raingauge 0367601W overestimates the average merged values for Sub-catchment 26, could be attributed to either the size of the catchment or the altitude of the raingauge (1672 m), which is higher than most parts of the sub-catchment, (average=1645 m).

The correlation between selected daily raingauges and the average merged rainfall depth for the sub-catchments in the Liebenbergsvlei catchment were found to be generally good, with correlation coefficients greater than 0.5 for most sub-catchments. However, in most of the cases the use of a daily raingauge to represent the rainfall for a sub-catchment overestimates the average areal rainfall depth of the sub-catchment by between 5% and 50%. The relationships obtained are largely dependent on the spatial variation of rainfall over the sub-catchments and the location and altitude of the daily raingauges relative to the catchment. An ideal perfect relationship ($Y = x$; $R^2 = 1$) between the daily raingauges and average rainfall depths of the sub-catchments would only be obtained under a condition of perfect spatial uniformity of daily rainfall over the sub-catchments.

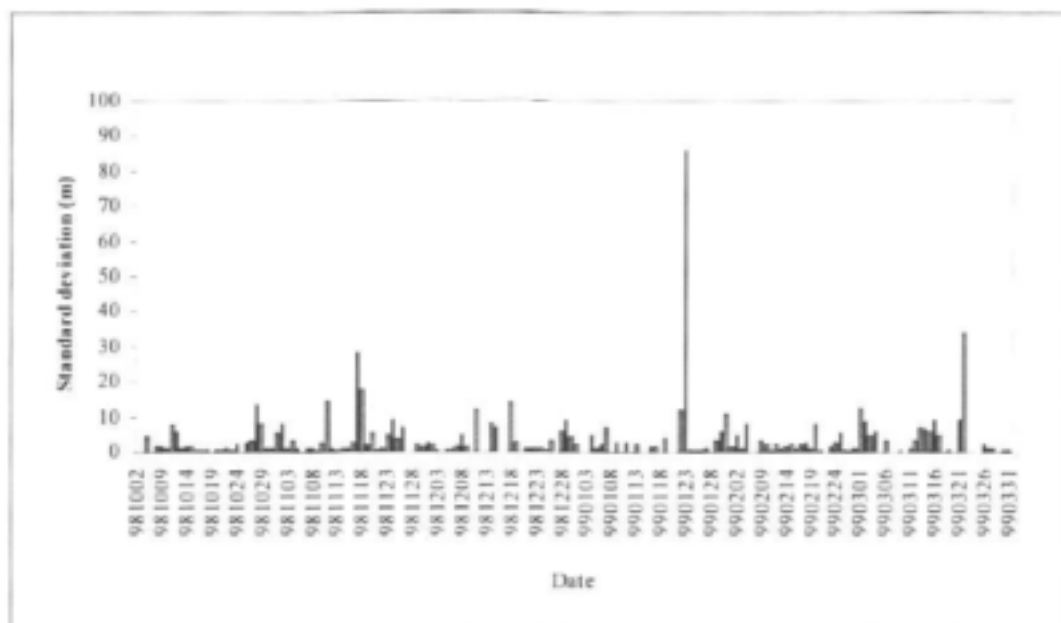


Figure 3.31 Standard deviation of the spatial distribution of daily rainfall within Sub-catchment 26

As shown in Table 3.8, in most of the sub-catchments the point rainfall data represent the areal rainfall of the sub-catchments reasonably well and the gauged rainfall generally overestimates the mean areal sub-catchment rainfall. Sub-catchment 1 has very small area (6.26 km²) and there is no raingauge located inside the perimeter of the sub-catchment. The spatial distribution of rainfall is relatively high as the catchment is located in the higher altitude part of Liebenbergsvlei Catchment. As a consequence, none of the raingauges located around the sub-catchment satisfactory represent the mean areal rainfall of the sub-catchment. Sub-catchments 11, 12 & 13 are located in the very mountainous part of the Liebenbergsvlei Catchment and are highly susceptible to ground clutter of the radar image.

In most of the sub-catchments, excluding Sub-catchments 1, 11, 12 and 13, rainfall measured by raingauges located in or near to the sub-catchments generally represent the mean areal sub-catchment rainfall reasonably well. However, the representation of the mean areal rainfall of the sub-catchment by a raingauge varies between sub-catchments (Frezghi, 2005).

3.4.5. Conclusions from Merging Radar and Raingauge Data

Merged rainfall fields for the Liebenbergsvlei Catchment were generated using an algorithm developed by Sinclair (2004), which is based on the conditional merging technique of Ehret (2002). In this study, the merging technique was validated against data from tipping bucket raingauges used in conditioning of the radar images. The conditional merging technique is intended to retain the rainfall depths used in the conditioning of the radar images in the merged rainfall field. The results obtained indicated that gauged rainfalls at the conditioning raingauges were not always retained as the merging technique developed by Sinclair (2004) masks the area where the radar did not register any rain, even though raingauges in this area may have reported rainfall. This was done to avoid false rainfall in other parts of the area. When the masked values were removed from the comparison, a nearly perfect relationship

was obtained between the conditioning raingauge data and the merged pixel rainfall values located at the conditioning raingauge.

Table 3.8 Summary of linear regressions between mean areal rainfall of the sub-catchments and point rainfall data

Sub-catchment	Tipping bucket Raingauge	Daily Raingauge	X- coefficient	Correlation (R^2)
1	L008		0.6303	0.3839
2	L008		0.7618	0.5256
	L013		0.7961	0.6984
		0332284W	0.8308	0.5810
3	L013		0.7202	0.5616
4	L012		0.8125	0.8045
		0332104W	0.7438	0.7467
5	L012		0.6996	0.6505
		0332104W	0.6857	0.7061
		0332073W	0.6653	0.6185
6	L021		0.7523	0.7270
		0332066W	0.8803	0.4809
7	L016		0.9388	0.6962
8	L017		1.0558	0.9058
		0332066W	1.2578	0.8219
		0332098W	0.9615	0.7558
9	L016		0.4337	0.8120
		0331845W	0.5124	0.7141
10		0367802W	1.0615	0.7830
14	L001		0.7937	0.7137
15	L010		0.9056	0.7942
16	L015		1.0820	0.7021
17	L045		0.9365	0.8074
18	L045		1.0551	0.7073
19	L004		0.9679	0.8135
20	L005		0.9104	0.7474
		0331590W	0.8578	0.7683
21	L009		1.3507	0.8561
		0331585W	1.0545	0.7723
22	L022		0.6660	0.5685
	L019		0.8677	0.6686
		0331607W	0.9176	0.7217
23				
24	L031		0.6626	0.7865
	L038		0.7781	0.6641
		0367462W	0.7318	0.6247
25	L032		0.7884	0.8740
		0367768W	0.7330	0.6519
26	L036		0.7103	0.8113
	L042		0.8230	0.8996
		0367666W	0.7696	0.7659

The merging technique was independently verified using daily raingauges which were not used in the conditioning of the radar images. For most sub-catchments reasonably good verifications were obtained with X-coefficient ranges between 0.8 to 1.2 and correlation coefficients greater than 0.5. However, the relationship was noted by Frezghi (2005) to depend on the distance between the tipping bucket raingauge used in the conditioning of the radar image and the daily raingauge under consideration.

The average merged rainfall values for each sub-catchment of the Liebenbergsvlei catchment were related to rainfall from raingauges selected to represent rainfall in the sub-catchments. The relationships were generally found to be good, with correlation coefficients of greater than 0.5 for most of the sub-catchments. However, the raingauges selected to represent the areal rainfall of the sub-catchments generally overestimated the mean areal merged rainfall values of the sub-catchments by between 5% and 50%. The relationship developed and the historical rainfall data from the raingauges can be used to provide improved estimates of average catchment rainfall for use in modelling and other hydrological studies. The errors in estimating rainfall for a catchment using a raingauge have been highlighted in this study and the need to use the merging process where radar data is available is evident.

3.5. String of Beads Model

Stochastic hydrology is used to generate synthetic streamflow or rainfall sequences that are statistically similar to observed streamflow or rainfall sequences. Statistical similarity implies sequences that have statistics and dependence properties similar to those of the historical record. Therefore, stochastic rainfall generated by rainfall models is designed to mimic the statistics of the observed rainfall data. Stochastic rainfall series should also mimic the observed spatial and temporal variability of rainfall in a catchment and the synthetic rainfall values may be generated at finer spatial and temporal details than the observations (Siriwardena *et al.*, 2002)

Continuous simulation modelling of the hydrologic system requires long-term, high-resolution climate data and, with the continuing advances in stochastic rainfall models, continuous simulation is rapidly becoming a practical tool for hydrological risk assessment and hydraulic engineering designs (Kuczera and Coombes, 2002). In this study, the String of Beads model (Pegram and Clothier, 2002) was selected to generate stochastic rainfall values. The model was selected for evaluation because it was readily available, developed in South Africa and verified using data from the Liebenbergsvlei catchments and is able to produce rainfall values at 1 km x 1 km and 5 minutes spatial and temporal resolutions respectively.

The String of Beads Model (SBM) is a high-resolution space-time model of radar rainfall images. It is a stochastic model that takes advantage of the detailed spatial and temporal information captured by weather radar and combines it with the long term seasonal variation captured by a network of daily raingauges. Pegram and Clothier (2002) modelled the alternating wet-dry process, or event arrival and duration, as a one-dimensional process, while the detailed wet process is modelled as a three-dimensional (two space and one time) process at 1 km and 5 minute spatial and temporal resolutions respectively, over an area of 16000 km², which is consistent with the images from the observed radar data. The three-dimensional rainfall events distributed on a one-dimensional time line is analogous to a "String of Beads" (Pegram and Clothier, 2002).

3.5.1. Data Representation

Synthetic rainfall values were generated using the SBM, calibrated using the Bethlehem radar and raingauge data from the Liebenbergsvlei Catchment, which was made available by Clothier (2004). The model provided generates rainfall images with integer values and generates real (floating point) Image Mean Flux (IMF) or average rainfall rate values at the image scale. After calibration, the input required for the String of Beads model simulation is the duration of the simulation, the date when simulation starts, the simulation stop date and random seeds. The rainfall image produced by the String of Beads model has a 64 km radius and the model configuration used in this study was calibrated by Clothier (2004) for a study area near Bethlehem (Figure 3.21). The model can generate rainfall images at increments of 5 minutes, 1 hour, daily, monthly and annual totals. However, in this study only daily, monthly and annual images were produced.

3.5.2. Evaluation of the SBM

Daily values were extracted from the generated images at pixel size and image scale to compare the statistics with the available historical raingauge data. IMF or rainfall values at the image scale for all time scales (daily, monthly and yearly) are output after the simulation, while values at pixel size were extracted using ArcGIS software. All the simulated images were converted to grids and all values were extracted using ArcInfo tools (Frezghi, 2005).

3.5.2.1. Annual Statistics

At the image scale, the statistics of a 50-year period of generated rainfall was compared against statistics computed from the observed rainfall data at 54 raingauges located in the study area. Table 3.9 contains the statistical comparison between the two rainfall series.

Table 3.9 Comparison between SBM generated and observed annual rainfall statistics at the image scale

Series	Mean (mm)	Standard deviation (mm)	Coefficient of skewness	Kurtosis	Lag-one auto-correlation	Mean annual number of wet days
Generated	545.2	90.7	-0.034	-0.56	-0.12	198
Observed	654.0	108.0	0.300	-0.03	0.17	313

The generated rainfall images underestimate the mean annual rainfall and standard deviation by 17% and 16% respectively. The differences in the coefficients of skewness, which are a measure of the degree symmetry in the distribution of a variable, implies that the observed rainfall values are skewed above the mean observed rainfall while the generated rainfall values are more symmetric than the observed with rainfall data with only a slightly negative skewed distribution. Kurtosis is a measure of degree of "peakedness" or flatness in the variable distribution and both the observed and generated distributions have a flatter distribution than a normal distribution. The results in Figure 3.32 show that average annual rainfall total of the 54-rain gauges for a 50 year period of record from 1948 to 1997 have values ranging from slightly over 900 mm to just below 500 mm.



Figure 3.32 Observed average annual total rainfall (Pegram and Clothier, 2002)

The average annual rainfall totals of the generated rainfall images for a 50 year period of records are shown in Figure 3.33. The maximum value is 745 mm and the minimum value is 336 mm. The serial correlations of the generated and observed annual series indicate that the structure of the generated annual series is poor. Other significant difference between the two series of annual rainfall totals is that there are no values exceeding 800 mm in the generated rainfall images and there are values which are less than 400 mm, which do not occur on the observed series.

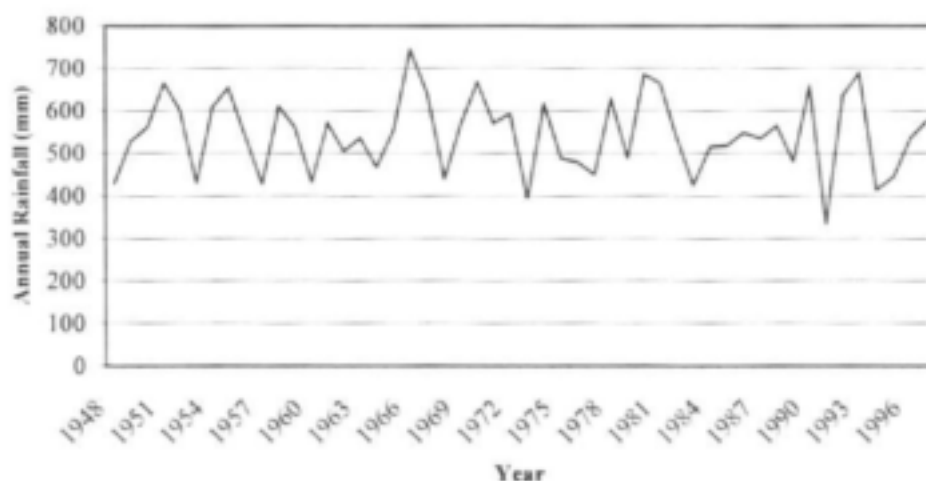


Figure 3.33 Average annual total rainfalls generated using the SBM

At the pixel scale, raingauges were selected inside the Liebenbergsvlei catchment and compared with a pixel rainfall value from generated rainfall images at the same position as the raingauges. Table 3.10 contains a summary of the statistics of the generated and observed rainfall series.

Table 3.10 Statistical comparison between SBM generated and observed annual rainfall at pixel scale

Raingauge	Data	Mean (mm)	Standard deviation (mm)	Coefficient of Skewness	Kurtosis	Lag-one auto-correlation	Mean annual number of wet days
0297721W	Generated	506.9	129.8	1.16	1.68	-0.008	79.5
	Observed	720.4	163.7	0.26	-1.11	0.098	102.8
0331271W	Generated	514.7	129.6	0.04	-0.15	-0.174	78.2
	Observed	641.8	152.1	0.53	0.09	-0.011	86.4
0331385W	Generated	506.9	132.6	0.87	0.45	-0.270	80.4
	Observed	673.5	129.1	0.14	-0.84	0.074	129.6
0331402W	Generated	511.4	141.2	0.36	-0.20	-0.238	78.6
	Observed	630.3	112.9	0.21	-0.47	0.164	115.4
0331455W	Generated	542.6	133.8	0.33	-0.15	0.016	78.7
	Observed	720.6	168.0	0.53	-0.24	0.285	107.8
0331467W	Generated	507.9	145.5	0.87	0.31	-0.141	79.9
	Observed	626.7	128.2	0.59	-0.30	0.276	129.4

In all cases the generated mean rainfall at the pixel scale underestimated the observed mean annual raingauge values. The observed rainfall data are more variable than the values generated by the SBM. Although both the selected raingauge data and the generated rainfall series at a pixel have positive coefficients of skewness, the values are not the same. The distribution of the generated annual rainfall series at the image scale is generally flatter than the observed rainfall data from the 54-raingauges, while at a pixel scale the generated rainfall values are more variable in their degree of peakedness. In general, the distribution of observed rainfall data in the Liebenbergsvlei Catchment are not consistent in terms of their peakedness with some raingauges have sharper peaks than a normal distribution and flatter peaks. Similarly, the generated rainfall values at the pixel scale are also not consistent in terms peakedness. It is evident from the results in Table 3.10 that the serial correlation of the annual totals is not well represented in the generated series and that the mean annual number of wet days is consistently underestimated in the generated series.

3.5.2.2. Monthly Statistics

Monthly totals are the next temporal scale considered and a statistical comparison was made between the generated and observed rainfall series. Table 3.11 summarises the statistical comparison between the two rainfall series at the image scale.

Table 3.11 Statistical comparison between SBM generated monthly rainfall and observed monthly rainfall at image scale

Statistic	Series	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean Monthly (mm)	Generated	100.60	72.50	69.40	46.00	11.70	2.30	4.00	4.70	20.60	50.90	75.90	89.20
	Observed	105.00	82.00	78.00	50.00	22.00	8.00	8.00	16.00	30.00	70.00	88.00	99.00
Standard Deviation (mm)	Generated	46.60	32.3	28.7	20.20	9.80	4.90	6.80	6.60	15.50	27.80	33.40	35.40
	Observed	42.00	39.00	33.00	27.00	22.00	10.00	14.00	21.00	39.00	38.00	37.00	36.00
Coefficient of Skewness	Generated	3.20	2.54	2.15	1.49	0.35	0.09	0.12	0.16	0.67	1.69	2.53	2.86
	Observed	1.00	1.00	1.50	0.50	2.00	2.50	1.75	1.60	3.40	1.00	0.50	0.25
Lag-one auto-correlation	Generated	0.67											
	Observed	0.69											

Both the mean monthly and standard deviation of the observed rainfall is reasonably well generated by the SBM, particularly for the wetter months. However, the generated series consistently underestimated the mean monthly observed rainfall data. The smaller differences in the monthly lag-one serial correlations show that the SBM reproduced the monthly rainfall statistics better than the annual statistics. The monthly rainfall statistics from 54 raingauges over the period of 50 years are shown in Figure 3.34.

There is a significant seasonal variation in the rainfall at Bethlehem. Of the dryer months, it is apparent that September is prone to occasional large monthly rainfall totals. The mean of 29.7 mm, standard deviation of 39.5 mm, skewness of 3.4 and kurtosis in excess of 13.3 for September are reduced to 22.6 mm, 18.7 mm, 1.4 and 1.9 respectively if the two extreme events of 1957 and 1987 are excluded (Pegram and Clothier, 2004). The generated monthly rainfall distribution for a 50 years period are shown in Figure 3.35, which is the equivalent graph to Figure 3.34 generated by Clothier (2004). As shown in Figure 3.35, the generated mean monthly rainfall values follow the statistical trend of the observed monthly rainfall distribution. However, the other statistics do not follow the statistics of the observed monthly rainfall values as well.

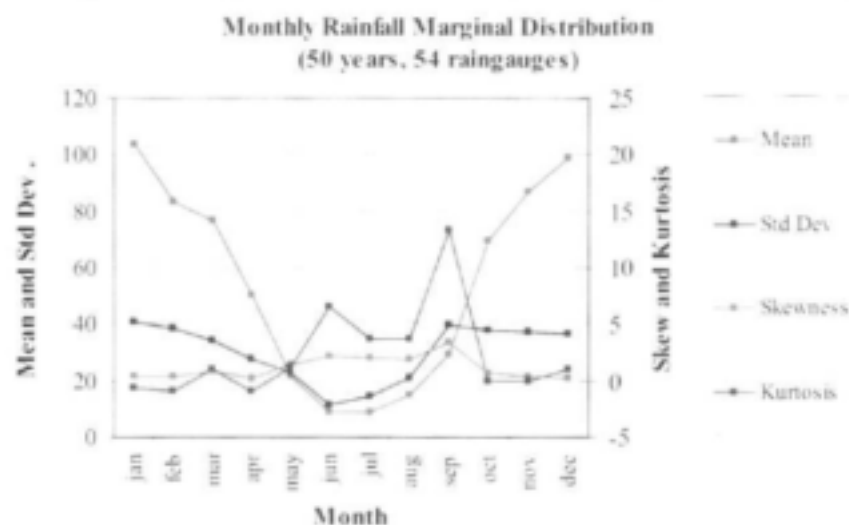


Figure 3.34 Observed monthly rainfall distribution over 50 year period (after Clothier, 2004)

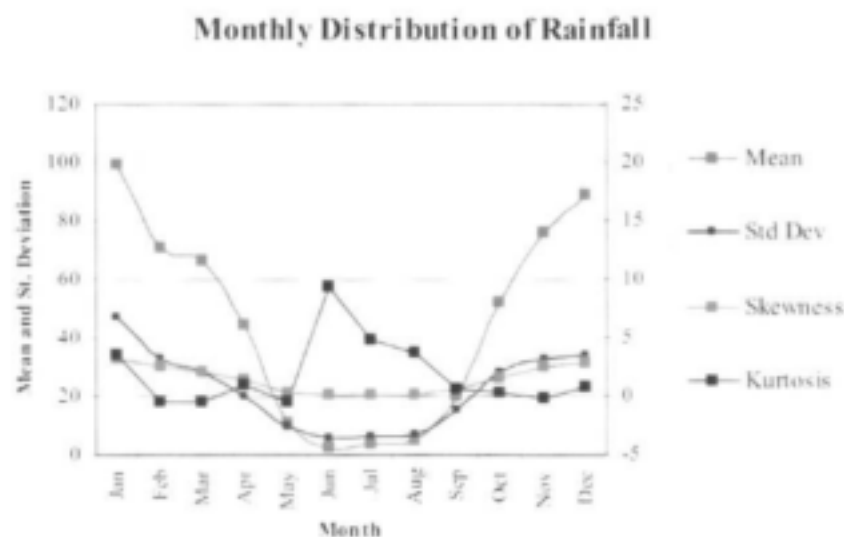


Figure 3.35 SBM generated monthly rainfall distribution over 50 year period

3.5.2.3. Daily Statistics

Figure 3.36 contains the frequency distribution for the observed average daily rainfall in the study area. In Figure 3.36, five series are plotted, one for each season and one for the combined analysis considering all days, independent of the season. The dry winter season is clearly defined with a probability of observing any rain in the study area of 21%. By contrast, the probability of receiving any rain in the study area on a summer day is 88 %. Autumn and spring probabilities are between those of summer

and winter, as expected. Spring shows a larger probability of high daily rainfall than autumn over most of the range, but particularly between the 98.0 and 99.5 percentiles (Pegram and Clothier, 2002). The frequency distribution of the average generated daily rainfall values in the study area are shown in Figure 3.37. For the generated series there is a 10 % probability for any rain during the dry winter season. Moreover, the probability of receiving rainfall in the summer season is 75 %, which indicates that the probabilities of generated rainfall occurring for any rainfall amount are less than the probabilities in the observed rainfall series.

Table 3.12 contains a summary of statistics computed from the observed and generated daily rainfall series over a 50 year period at the image scale. The SBM is a short duration rainfall model and is expected to give a better result on daily basis than on monthly or annual periods. The generated mean rainfall values reproduce the observed rainfall mean reasonably well. However, as shown in Table 3.12, Figure 3.38 and Figure 3.39, the SBM generates fewer wet days over the 50 year period than the number of wet days in the observed data and assigns more rainfall to the wet days. Unlike the case for annual periods, the lag-one autocorrelations computed from the generated and observed daily series are similar.

Table 3.12 Statistical comparison between SBM generated daily rainfall and observed daily rainfall at the image scale

Statistic	Series	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean Daily Rainfall (mm)	Generated	3.24	2.45	2.24	1.53	0.38	0.08	0.13	0.15	0.69	1.64	2.53	2.88
	Observed	3.39	2.90	2.52	1.67	0.71	0.27	0.26	0.52	1.00	2.26	2.93	3.19
Mean Daily Rainfall on Wet Days (mm)	Generated	11.64	11.47	11.76	11.55	10.41	12.91	12.35	9.31	10.64	11.42	11.37	11.53
	Observed	8.85	8.00	8.97	8.33	8.15	9.30	8.00	10.00	10.48	9.59	8.31	8.61
Number of Wet days	Generated	431	302	295	199	56	9	17	25	97	223	334	387
	Observed	593	512	435	300	135	43	50	80	143	365	529	575
Lag-one Auto-correlation	Generated	0.680											
	Observed	0.714											

The number of dry days in the generated 50 years series follows the trends in the observed data, although the numbers are not exactly the same. The SBM generated almost the same number of dry days in April to June, underestimated the number of dry days during winter (July-September) and overestimated the number of dry days during the summer months (October – March).

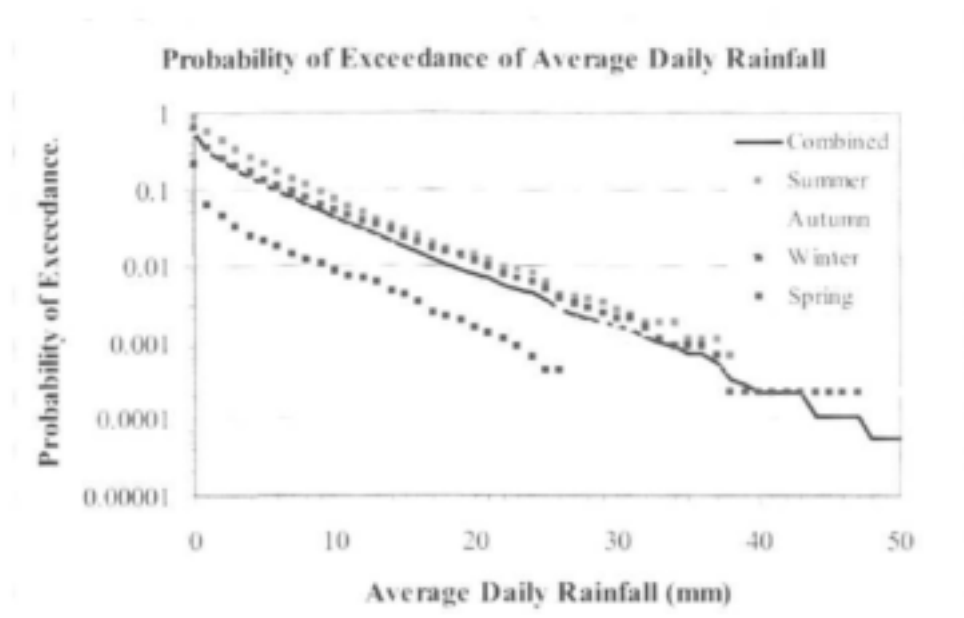


Figure 3.36 Probability of exceedance of average observed daily rainfall over a 50

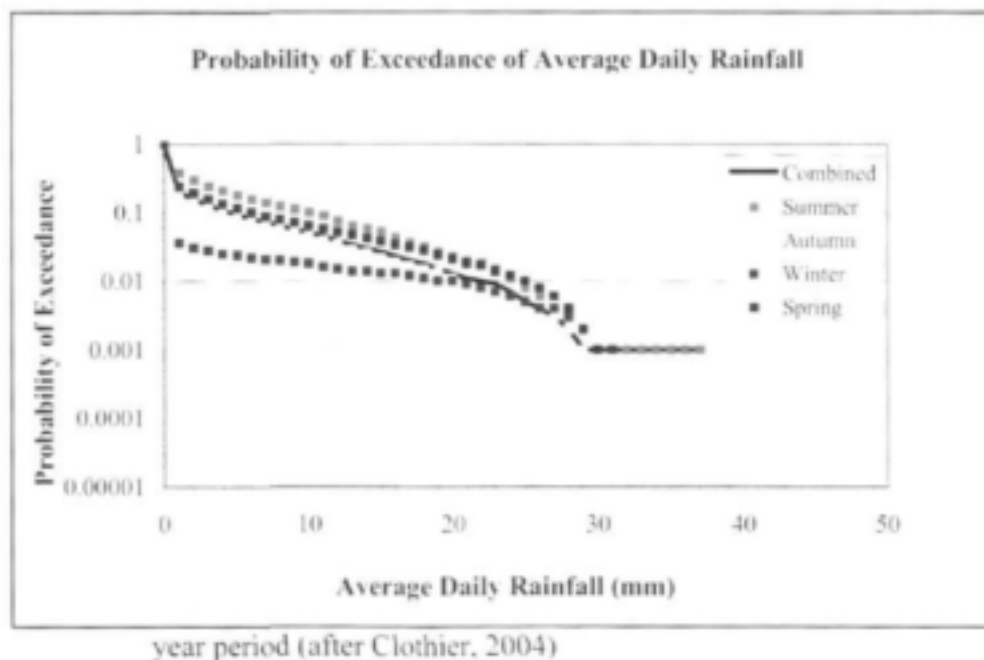


Figure 3.37 Probability of exceedance of average generated daily rainfall over a 50 year period

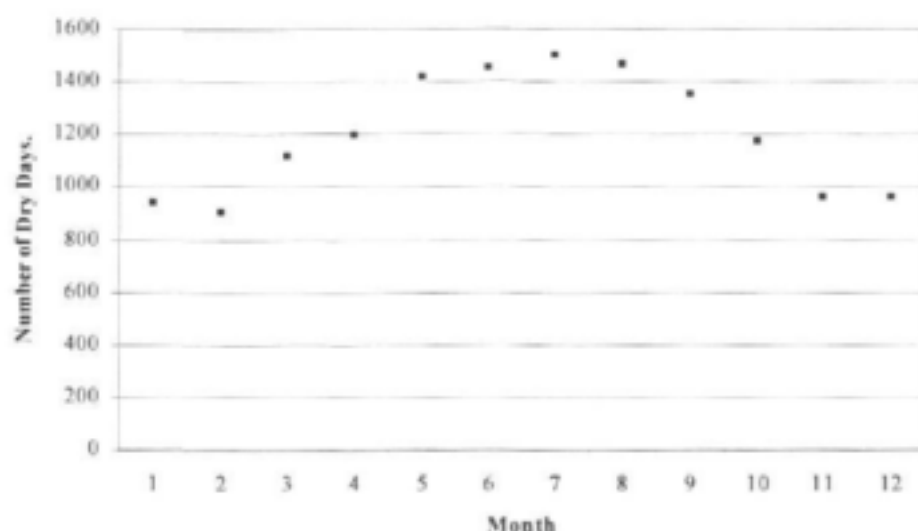


Figure 3.38 Number of observed dry days for each month over the 50 year period (after Clothier, 2004)

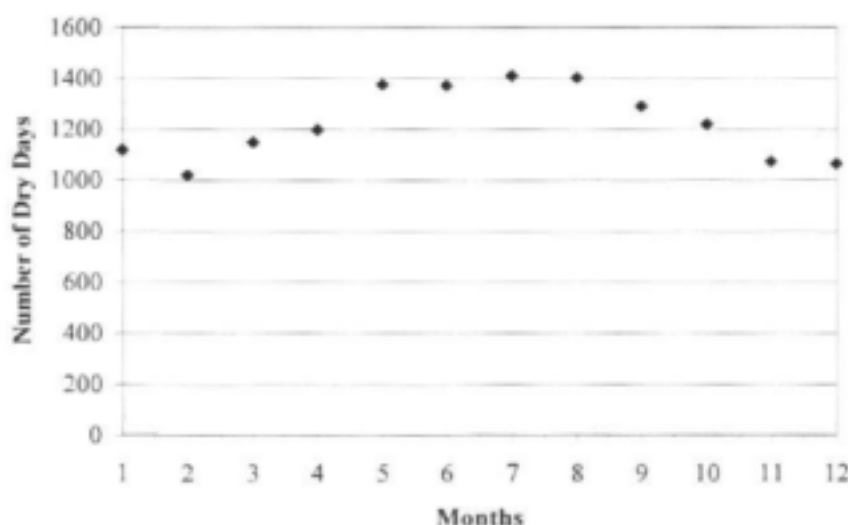


Figure 3.39 Number of generated dry days for each month over 50 year period

3.5.3. Assessment of the Performance of the SBM

Generally, the SBM reproduced the observed statistics at a daily time scale reasonably well and better than at monthly or annual time scales. The SBM is a short duration rainfall model designed to mimic rainfall values at a detailed temporal and spatial resolution. Hence, small errors at 5 minutes durations accumulate over longer durations to the errors evident at the daily and longer time scales. Spatially, the SBM reproduced the statistics of the selected raingauges considered in this study. The

spatial structure of the SBM is designed on the basis of radar images for a 4 year period (1996- 2000) and the observed rainfall from 54 raingauges, while the temporal structure is based solely on the observed rainfall data from the 54 raingauges. Therefore, errors associated with the radar images, compared to the raingauges, may influence the generated rainfall values at the pixel scale over the 50 year period.

In this section, it was shown that the SBM could reproduce rainfall values better at daily time scales than at monthly or annual periods. Therefore, it is concluded that an appropriately calibrated SBM may be used in rainfall-runoff modelling which requires rainfall at detailed spatial and temporal resolutions. When the rainfall model is required at monthly or annual time steps, it is advisable to use a stochastic model which is designed for monthly and annual time steps, for example the stochastic generation of annual rainfall series as developed by Srikanthan and McMahon (2001).

3.6. Flood Routing in Ungauged River Reaches

When modelling large catchments with the *ACRU* model, catchments are generally subdivided into sub-catchments, each with its own rainfall and catchment characteristics. Runoff from each sub-catchment is routed through downstream river reaches to the catchment outlet. The Muskingum method of flow routing in river reaches is used in *ACRU* (Smithers and Caldecott, 1993). However, without observed flow data, the Muskingum K and X parameters are difficult to estimate. The objective of this component of the study, undertaken by Tewolde (2005), was to assess Muskingum-based methods for flow routing in ungauged river reaches, both with and without lateral inflows.

3.6.1. Introduction

As defined by Fread (1981) and Linsley *et al.* (1982), flood routing is a mathematical method for predicting the changing magnitude and celerity of a flood wave as it propagates down rivers or through reservoirs. Numerous flood routing techniques, such as the Muskingum flood routing methods, have been developed and successfully applied to a wide range of rivers and reservoirs (France, 1985). Flood routing methods are generally categorised into two broad, but somewhat related applications, namely reservoir routing and open channel routing (Lawler, 1964). These methods are frequently used to estimate inflow or outflow hydrographs and peak flow rates in reservoirs, river reaches, farm ponds, tanks, swamps and lakes (NRCS, 1972; Viessman *et al.*, 1989; Smithers and Caldecott, 1995).

Flood routing is important in the design of flood protection measures in order to estimate how proposed measures will affect the behaviour of flood waves in rivers and thus enable adequate protection and economic solutions to be found (Wilson, 1990). In practical applications, the prediction and assessment of flood level inundation involves two steps. A flood routing model is used to estimate the outflow hydrograph by routing a flood event from an upstream flow gauging station to a downstream location. Then the flood hydrograph is input to a hydraulic model in order to estimate the flood levels at the downstream site (Blackburn and Hicks, 2001).

Flood routing procedures may be classified as either hydrological or hydraulic (Choudhury *et al.*, 2002). Hydrological methods use the principle of continuity and a relationship between discharge and the temporary storage of excess volumes of water during the flood period (Shaw, 1988). Hydraulic methods of routing involve the numerical solutions of either the convective diffusion equations or the one-

dimensional Saint-Venant equations of gradually varied unsteady flow in open channels (France, 1985).

As noted by US Army Corps of Engineers (1994), several factors should be considered when evaluating which routing method is the most appropriate for a given situation. The factors that should be considered in the selection process include, *inter alia*, backwater effects, floodplains, channel slope, hydrograph characteristics, flow network and subcritical and supercritical flow (US Army Corps of Engineers, 1994). The selection of a routing model is also influenced by other factors such as the required accuracy, the type and availability of data, the available computational facilities, the computational costs, the extent of flood wave information desired, and the familiarity of the user with a given model (NERC, 1975; Fread, 1981).

Hydraulic methods generally describe the flood wave profile more adequately when compared to hydrological techniques, but practical application of hydraulic methods are restricted because of their high demand on computing technology, as well as on quantity and quality of input data (Singh, 1988). Even when simplifying assumptions and approximations are introduced, the hydraulic techniques are complex and often difficult to implement (France, 1985). Studies have shown that the simulated outflow hydrographs from hydrological routing methods always have peak discharges higher than those of the hydraulic routing methods (Haktanir and Ozmen, 1997). However, in practical applications, the hydrological routing methods are relatively simple to implement and reasonably accurate (Haktanir and Ozmen, 1997). An example of a simple hydrological flood routing technique used in natural channels is the Muskingum flood routing method (Shaw, 1988).

Among the many models used for flood routing in rivers, the Muskingum model has been one of the most frequently used tools, because of its simplicity (Tung, 1985). As noted by Kundzewicz and Strupczewski (1982), the Muskingum method of flood routing has been extensively applied in river engineering practices since its introduction in the 1930s. The modification and the interpretation of the Muskingum model parameters in terms of the physical characteristics, extends the applicability of the method to ungauged rivers (Kundzewicz and Strupczewski, 1982). Most catchments are ungauged and thus a methodology to compute the flood wave propagation down a river reach or through a reservoir is required. One option is to calibrate flood routing models on gauged catchments and relate their parameters to physical characteristics of the catchment and channel (Kundzewicz, 2002). Flood routing models with derived parameters then can be applied to ungauged catchments in the region (Kundzewicz, 2002).

In this study, the Muskingum-Cunge method (Cunge, 1969) is adopted to estimate the model parameters because of its simplicity as well as its ability to perform flood routing in ungauged catchments by estimating the model parameters from flow and channel characteristics. The Muskingum-Cunge parameter estimation method utilises catchment variables such as flow top width (W), slope (S), average velocity (V_{av}), reference discharge (Q_0), celerity (V_w), and river length (L) to estimate the parameters of the Muskingum method.

When performing flood routing in ungauged catchments, the model parameters have to be estimated without observed hydrographs. The inflow hydrograph could, for example, be generated using a hydrological model such as the *ACRU* model (Schulze, 1995). For this component of the study, observed inflow hydrographs are used as input to simulate outflow hydrographs.

The objective of this component of the study is to assess the performance of the Muskingum-Cunge method in ungauged catchments using parameters derived using variables estimated both from empirical equations developed for different river reaches and using variables estimated from selected (assumed) cross-sections within the river reach. The study was performed using data from the Thukela Catchment in South Africa.

3.6.2. Methodology

This section details the flood routing methods applied to selected events. At ungauged sites the Muskingum-Cunge equation was used, with flow variables estimated using both an empirical approach (MC-E) and using variables computed from a user-defined river reach cross section (MC-X). The computed outflow hydrographs were computed and compared graphically and statistically with the observed hydrographs. The methodology is detailed as follows:

- The Muskingum K and X parameters were estimated for each reach on the basis of empirically determined flow characteristics such as flow top width (W), wetted perimeter (P), flow depth (y), hydraulic radius (R) and average velocity (V_{av}). The method is referred to as MC-E.
- The Muskingum K and X parameters were estimated for each reach on the basis of assumed (selected) channel cross-sectional shape and dimensions, as observed in the field, such as maximum flow depth (y) and maximum flow top width (W). The method is referred to as MC-X.
- Outflow hydrographs computed using the estimated parameters were compared to the observed hydrographs both statistically as well as visually with regard to flood volume, the magnitude and timing of peak discharge as well as the shape of the hydrograph.

The details of the calculation steps are explained in the following sections.

3.6.2.1. The MC-E Method

The depth of flow and discharge in a reach can be estimated from empirical relationships recommended by the US Reclamation Service (Chow, 1959). Hence, the Muskingum K and X parameters can be estimated in ungauged river reaches using inflow hydrographs and channel dimensions estimated from empirical relationships. In this study, the observed inflow hydrographs obtained from the Department of Water Affairs and Forestry (2003) and empirically estimated channel variables were used to estimate the Muskingum K and X parameters.

The Muskingum K parameter is estimated from Equation 3.3 as follows (Chow, 1959; Fread, 1993):

$$K = \frac{\Delta L}{V_w} \quad \dots \text{Eq. 3.3}$$

where

- K = wave travel time (s),
- ΔL = reach length (m), and
- V_w = celerity (m.s^{-1}).

For a parabolic cross-section, the celerity (V_w) may be estimated using the relationships in Table 3.13 and Equation 3.4 (Viessman *et al.*, 1989):

$$V_w = \frac{11}{9} V_{av} \quad \dots \text{Eq. 3.4}$$

where

$$V_{av} = \text{average velocity (m.s}^{-1}\text{)}.$$

Table 3.13 Estimation of celerity for various channel shapes (Viessman *et al.*, 1989)

Channel shape	Manning equations	Chezy equation
Wide rectangular	$5/3 V_{av}$	$3/2 V_{av}$
Triangular	$4/3 V_{av}$	$5/4 V_{av}$
Parabolic	$11/9 V_{av}$	$7/6 V_{av}$

The average velocity (V_{av}) is calculated from the Manning Equation 3.5 as follows (Chow, 1959):

$$V_{av} = \frac{1}{n} R^{2/3} \sqrt{S} \quad \dots \text{Eq. 3.5}$$

where

n = manning roughness coefficient (dimensionless) which may be estimated using the method outlined by (Chow, 1959),

R = hydraulic radius (m), and

S = slope (m/m).

From Manning's equation, a "section factor", can be calculated as (Chow, 1959):

$$AR^{2/3} = \frac{Q_0 n}{\sqrt{S}} \quad \dots \text{Eq. 3.6}$$

where

A = cross-sectional flow area (m^2).

Wilson and Ruffini (1988) defined a reference flow as :

$$Q_0 = Q_b + 0.5 (Q_p - Q_b) \quad \dots \text{Eq. 3.7}$$

where

Q_0 = reference flow ($\text{m}^3.\text{s}^{-1}$),

Q_b = minimum discharge ($\text{m}^3.\text{s}^{-1}$) and

Q_p = peak discharge ($\text{m}^3.\text{s}^{-1}$).

For a given hydrograph the reference flow Q_0 can be estimated using Equation 3.7. The roughness coefficient (n) and slope (S) were estimated from field visits and topographic maps respectively. The section factor in Equation 3.6 has to be estimated using either empirical relationship which relate discharge, depth and slope or by using charts that relate section factor to the depth of flow (Chow, 1959).

Equation 3.8 relates the wetted perimeter with discharge for natural rivers Lacey (1930; 1947; cited by Punmia and Pande, 1981):

$$P = c\sqrt{Q_0} \quad \text{for stable river channels} \quad \dots\text{Eq. 3.8}$$

where

P = wetted perimeter (m), and
 c = coefficient (between 4.71- 4.81).

The hydraulic radius (R) and hydraulic mean depth (d) can be assumed to be equal when the top flow width exceeds the mean flow depth by a factor of 20 m (Chow, 1959; Barfield *et al.*, 1981; cited in SAICEHS, 2001). Since the flow depth in most river reaches is small relative to the top width of a reach (W), the wetted perimeter (P) may be assumed to be equal to W .

The area of parabolic section is computed as (Chow, 1959; Koegelenberg *et al.*, 1997):

$$A = \frac{2yW}{3} \quad \dots\text{Eq. 3.9}$$

where

y = flow depth (m), and
 W = top flow width (m).

For wide parabolic channels, the hydraulic radius may be estimated from

Table 3.14 as:

$$R \equiv d = \frac{2y}{3} \quad \dots\text{Eq. 3.10}$$

where

d = mean flow depth (m).

Table 3.14 Hydraulic mean depth (Chow, 1959; Koegelenberg *et al.*, 1997)

Cross-section	Hydraulic mean depth (d)
Parabolic section	$(2/3) y$
Rectangular	y
Triangular	$0.5y$

Equations 3.9 and 3.10 can be substituted in Equation 3.6 and for a parabolic section results in:

$$AR^{2/3} = \left(\frac{2yW}{3}\right) \cdot \left(\frac{2y}{3}\right)^{2/3} = \frac{Q_0 n}{\sqrt{S}} \quad \dots\text{Eq. 3.11}$$

But P is assumed to be approximately equal to W and hence P can be substituted in Equation 3.11 to solve for y as shown in Equation 3.12

$$y = \left(\frac{Q_0 n}{0.508 P \sqrt{S}} \right)^{3/5} \quad \dots \text{Eq. 3.12}$$

For the MC-E approach, Equation 3.12 is used to estimate the depth of flow in ungauged reaches in order to estimate the Muskingum K and X parameters, as shown below.

3.6.2.2. The MC-X method

In the MC-X approach, the depth of flow in ungauged catchment is estimated by developing a rating curve for a selected (assumed) section in the reaches (Smithers and Caldecott, 1995). For the selected section a rating curve is developed based on the maximum width and depth relationships. Assuming a linear relationship between width and depth of the river section, for each given depth, a corresponding top width can be proportioned from the observed maximum depth and width ratio, as shown in Equation 3.13

$$W_i = \frac{W_{Max}}{y_{Max}} y_i \quad \dots \text{Eq. 3.13}$$

where

- W_i = top width (m),
- y_i = given depth (m),
- W_{Max} = maximum top width (m), and
- y_{Max} = maximum depth (m).

The wetted perimeter for each sub-section is calculated from geometrical equations (Chow, 1959; Koegelenberg *et al.*, 1997). For example, in the case of a parabolic section:

$$P_i = W_i + \frac{8y_i^2}{3W_i} \quad \dots \text{Eq. 3.14}$$

where

- P_i = wetted perimeter for given sub-section (m),
- W_i = top width for sub-sections (m), and
- y_i = given depth for sub-section (m).

The flow area can be estimated from the continuity equation or geometrical properties for a parabolic shape as follows (Chow, 1959; Koegelenberg *et al.*, 1997):

$$A = \frac{2yW}{3} \quad \dots \text{Eq. 3.15}$$

where

- A = flow area (m^2),
- W = top flow width (m), and
- y = flow depth (m).

The hydraulic radius is computed from the area and wetted perimeter as shown in Equation 3.16

$$R = \frac{A}{P} \quad \dots \text{Eq. 3.16}$$

The corresponding cumulative discharge is then calculated from the Manning equation as shown in Equation 3.17 .

$$Q = A \frac{1}{n} R^{2/3} \sqrt{S} \quad \dots \text{Eq. 3.17}$$

where

$$\begin{aligned} A &= \text{area (m}^2\text{), and} \\ Q &= \text{discharge (m}^3\text{.s}^{-1}\text{).} \end{aligned}$$

Since the roughness coefficients (n), hydraulic radius (R), flow area (A) and slope (S) of each reach is known, the discharge can be calculated from Equation 3.17 . Thus, from the derived rating curve, the depth of flow can be estimated for a given discharge and the corresponding width and wave celerity can be computed using Equation 3.13 and Table 3.13 respectively in the MC-X approach. Equations 3.3 and 3.19 are then used to estimate the K and X parameters respectively.

The average velocity calculated using Equation 3.5 is also used to calculate flow area of the reference flow from continuity equation in MC-E approach (Chow, 1959; Linsley *et al.*, 1988; KenBohuslay, 2004), or the area of flow can be estimated using the geometrical parameters as shown in the following equation for a parabolic section:

$$A = \frac{2yP}{3} \quad \dots \text{Eq. 3.18}$$

The wetted perimeter (P) and top flow width (W) are assumed to be equal.

The Muskingum K parameter is estimated using Equation 3.3 The top flow width ($W-P$), reference flow (Q_0), river slope (S), celerity (V_w) and sub-reach length (ΔL) are substituted in Equation 3.19 to estimate the Muskingum X parameter as follows (Chow, 1959; Fread, 1993):

$$X = \frac{1}{2} - \frac{Q_0}{2SPV_w \Delta L} \quad \dots \text{Eq. 3.19}$$

After the Muskingum K and X parameters have been estimated, the three routing coefficients (C_0 , C_1 , and C_2) are estimated using Equations 3.20 , 3.21 and 3.22 (Viessman *et al.*, 1989; Fread, 1993).

$$C_0 = (\Delta t + 2KX)/m \quad \dots \text{Eq. 3.20}$$

$$C_1 = (\Delta t - 2KX)/m \quad \dots \text{Eq. 3.21}$$

$$C_2 = (2K(1-X) - \Delta t)/m \quad \dots \text{Eq. 3.22}$$

$$m = 2K(1-X) + \Delta t \quad \dots \text{Eq. 3.23}$$

The three coefficients computed in Equations 3.20 to 3.22 are used in Equation 3.24 to estimate the discharge in successive time step (NERC, 1975):

$$Q_{j+1}^{t+1} = C_0 I_j^t + C_1 I_j^{t+1} + C_2 Q_{j+1}^t + C_3 \quad \dots \text{Eq. 3.24}$$

where

Q_j^t = outflow ($\text{m}^3 \cdot \text{s}^{-1}$) at time = t of the j -th sub-reach,

I_j^t = inflow ($\text{m}^3 \cdot \text{s}^{-1}$) at time = t of the j -th sub-reach, and

C_3 = lateral inflow term ($\text{m}^3 \cdot \text{s}^{-1}$), computed as shown in Equation 3.26

As suggested by Viessman *et al.* (1989), negative values of C_1 must be avoided. Negative values of C_2 do not affect the routed hydrographs. The negative values of C_1 can be avoided by satisfying Equation 3.25

$$\frac{\Delta t}{K} > 2X \quad \dots \text{Eq. 3.25}$$

After the X parameter has been determined, the routing time interval can be adjusted using the relationship shown in Figure 3.40 (Cunge, 1969; cited by NERC, 1975).

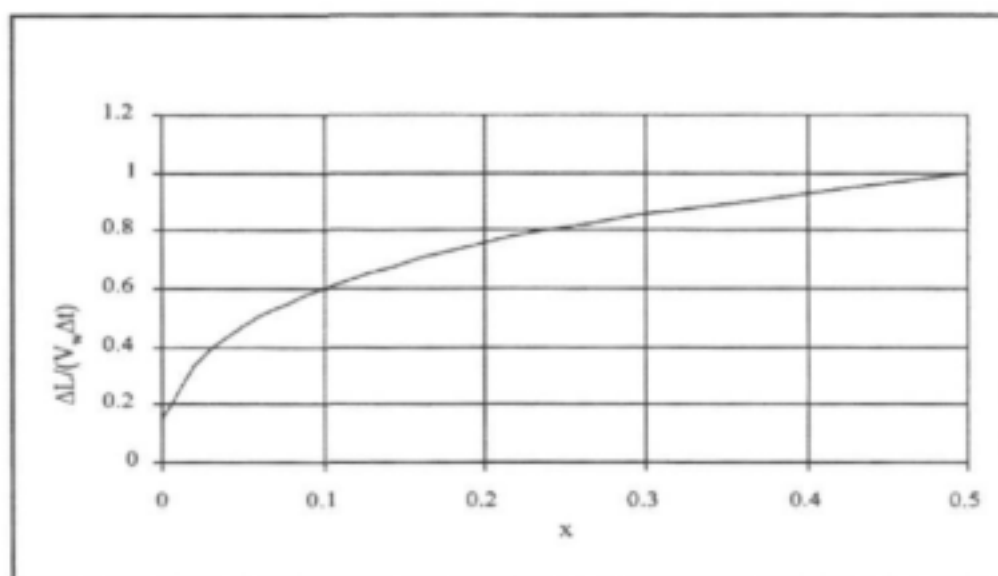


Figure 3.40 Cunge curve (Cunge, 1969; cited by NERC, 1975)

Since Q^t , I^t and I^{t+1} are known for a given time increment, Q^{t+1} is computed using Equation 3.24 and repeated for successive time increments to estimate the outflow hydrograph. In large catchments, where there is lateral inflow to the main stream, the volume of the outflow hydrograph may be larger than the inflow volume. Hence, lateral inflow should be considered and added to the main flow (NERC, 1975). The C_3 coefficient, calculated using Equation 3.26, is added as a lateral inflow term in Equation 3.24 (NERC, 1975):

$$C_3 = \frac{q \cdot \Delta t \cdot \Delta L}{2K(1 - X) + \Delta t} \quad \dots \text{Eq. 3.26}$$

where

- Δt = change in time (s),
- q = lateral inflow ($\text{m}^2 \cdot \text{s}^{-1}$) per unit length (m), and
- ΔL = reach length (m).

The lateral inflow per unit length at a specified time (q) is estimated from the Saint-Venant Equation. The Saint-Venant Equation for gradually varying flow in open channels is given as (NERC, 1975):

$$q = \frac{\Delta A}{\Delta t} + \frac{\Delta Q}{\Delta L} \quad \dots \text{Eq. 3.27}$$

Fread (1998), approximated the terms in Equation 3.27 as:

$$\frac{\Delta A}{\Delta t} \equiv \frac{\frac{A_j^{t+1} + A_{j+1}^{t+1}}{2} - \frac{A_j^t + A_{j+1}^t}{2}}{\Delta t} \quad \dots \text{Eq. 3.28}$$

$$\frac{\Delta Q}{\Delta L} \equiv \frac{\beta(Q_{j+1}^{t+1} - I_j^{t+1}) + (1 - \beta)(Q_{j+1}^t - I_j^t)}{\Delta L} \quad \dots \text{Eq. 3.29}$$

where

- β = weighing factor, which is between 0.5 – 1 (Fread, 1998), and
- A_j^t = cross sectional area [m^2] at time [t] of the j th sub-reach.

Hence, lateral flow per unit length at a specified time (q) can be calculated from Equation 3.30 and substituted in Equation 3.26

$$q = \frac{\frac{A_j^{t+1} + A_{j+1}^{t+1}}{2} - \frac{A_j^t + A_{j+1}^t}{2}}{\Delta t} + \frac{\beta(Q_{j+1}^{t+1} - I_j^{t+1}) + (1 - \beta)(Q_{j+1}^t - I_j^t)}{\Delta L} \quad \dots \text{Eq. 3.30}$$

When the ratio of lateral inflow to the main flow is too large, numerical difficulties in solving Equation 3.29 may arise. Increasing the routing length (ΔL) of the specified reach may solve the numerical difficulties (Fread, 1993).

From the continuity equation for an open channel (Chow, 1959; Linsley *et al.*, 1988; KenBohuslay, 2004):

$$Q = AV_w \quad \dots \text{Eq. 3.31}$$

The area term in Equation 3.30 can be substituted by discharge as shown in Equation 3.32 (Fread, 1998).

$$q = \frac{1}{V_w} \left(\frac{I_j^{t+1} + Q_{j+1}^{t+1}}{2} - \frac{I_j^t + Q_{j+1}^t}{2} \right) + \frac{\beta(Q_{j+1}^{t+1} - I_j^{t+1}) + (1-\beta)(Q_{j+1}^t - I_j^t)}{\Delta L} \quad \dots \text{Eq. 3.32}$$

According to Fread (1998), the value of β is between 0.5 and 1.0. A value of 0.7 was used in this study. Flow characteristics from the routed hydrograph such as the magnitude and timing of peak flow, hydrograph shapes and flow volume were compared to the characteristics of the observed events. The statistics used to assess the model performance are detailed in the following section.

3.6.2.3. Model Performance

As suggested by Green and Stephenson (1985), in order to compare a model output to observed data, criteria for making such a comparison must first be identified. Visual comparison by plotting simulated and observed hydrographs provides a valuable means of assessing the accuracy of the model output. However, visual comparisons usually tend to be subjective and need additional statistical analysis. To overcome these difficulties, as well as to highlight certain model peculiarities, statistical goodness-of-fit procedures were employed.

The difference in the observed and computed hydrograph were analysed by means of Root-Mean-Square Error (RMSE) and other goodness-of-fit statistics. A statistical goodness-of-fit procedure measures the deviation of simulated output from the observed input data set (Green and Stephenson, 1985).

Even though numerous goodness-of-fit criteria for assessing the accuracy of simulated output have been proposed, particular aspects may give more weight to selected output variables (Green and Stephenson, 1985). Hence, different goodness-of-fit statistics should be applied to assess different hydrograph components such as flood volume, hydrograph shape, peak flow magnitude and timing. Since an objective of this component of the study is to compute hydrographs in ungauged reaches, the criteria for assessment were selected as described below.

Equation 3.33 was used to estimate the actual errors in the computed hydrographs. The RMSE computes the magnitude of error in the computed hydrographs (O'Donnell, 1985; Schulze *et al.*, 1995).

$$RMSE = \sqrt{\frac{\sum_{i=1}^n (Q_{comp} - Q_{obs})^2}{n}} \quad i = 1, 2, 3, \dots, n \quad \dots \text{Eq. 3.33}$$

where

RMSE = Root-Mean-Square Error ($\text{m}^3 \cdot \text{s}^{-1}$ per event),

Q_{comp} = computed outflow ($\text{m}^3 \cdot \text{s}^{-1}$), and

Q_{obs} = observed outflow ($\text{m}^3 \cdot \text{s}^{-1}$).

As peak outflow is important in a single event model, a comparison of computed and observed peak flow rates, peak timing and volume were computed as shown in Equations 3.34 to 3.36 (Green and Stephenson, 1985).

$$E_{peak} = \frac{Q_{p-comp} - Q_{p-obs}}{Q_{p-obs}} 100 \quad \dots \text{Eq. 3.34}$$

$$E_{time} = \frac{t_{p-comp} - t_{p-obs}}{t_{p-obs}} 100 \quad \dots \text{Eq. 3.35}$$

$$E_{volume} = \frac{V_{comp} - V_{obs}}{V_{obs}} 100 \quad \dots \text{Eq. 3.36}$$

where

E_{peak} = peak flow error (%),

Q_{p-comp} = computed peak flows ($\text{m}^3 \cdot \text{s}^{-1}$),

Q_{p-obs} = observed peak flows ($\text{m}^3 \cdot \text{s}^{-1}$),

E_{time} = peak time error (%),

t_{p-comp} = time when Q_{comp} occurs (s),

t_{p-obs} = time when Q_{obs} occurs (s),

E_{volume} = peak volume error (%),

V_{comp} = computed total volume (m^3), and

V_{obs} = observed total volume (m^3).

Even though the RMSE, E_{peak} , E_{time} and E_{volume} statistics may appear to be reasonable, the shapes of the respective hydrographs may be different. Nash and Sutcliffe (1970), cited by Green and Stephenson (1985), proposed a dimensionless coefficient of model efficiency (E). The computed hydrograph is a better fit to the observed hydrograph when the coefficient of model efficiency (E) approaches 1 (Green and Stephenson, 1985). Hence, the hydrograph shape comparison was estimated using Equation 3.37

$$E = \frac{F_0^2 - F^2}{F_0^2} \quad \dots \text{Eq. 3.37}$$

where

$$F^2 = \sum_{i=1}^n [Q_{obs}(i) - Q_{comp}(i)]^2,$$

$$F_0^2 = \sum_{i=1}^n [Q_{obs}(i) - Q_m]^2, \text{ and}$$

Q_m = mean of the observed flows ($m^3.s^{-1}$).

The methodology outlined above was applied on selected sub-catchments in the Thukela Catchment.

3.6.3. Study Area

Three sub-catchments in the Thukela catchment were selected for analyses, with river lengths of 4, 21 and 54 km. The locations of the gauging weirs are shown in Figure 3.41. The characteristics of the selected reaches and variables calculated using both methods are contained in Table 3.15 and Table 3.16 for a selected event in each reach.

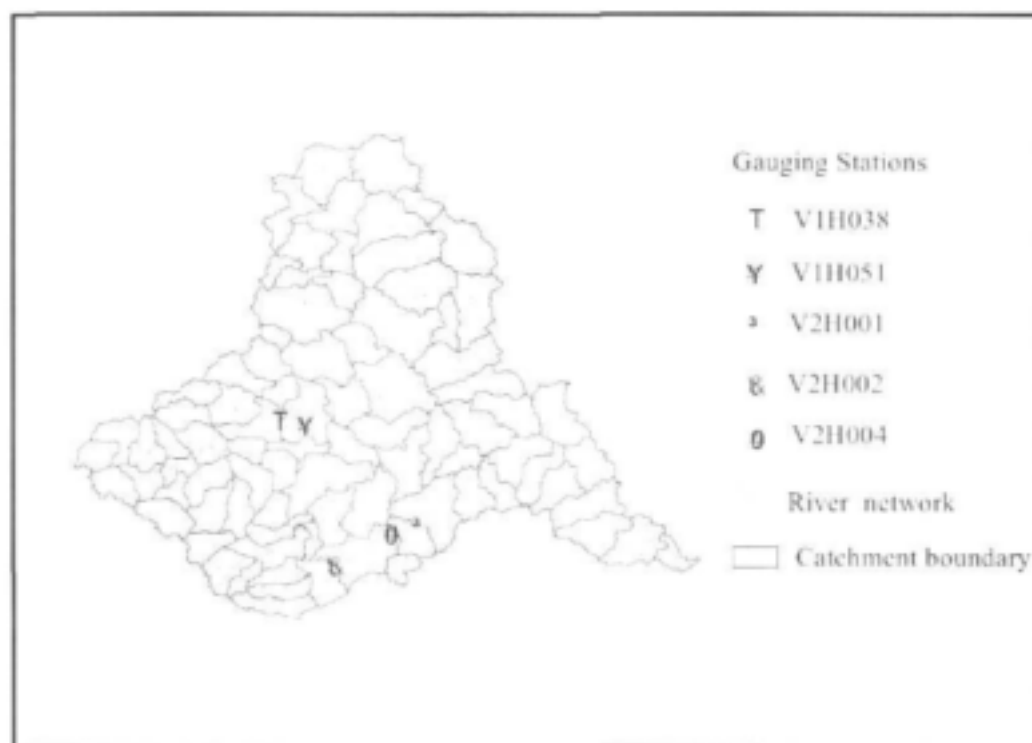


Figure 3.41 Selected gauging stations in the Thukela Catchment

Table 3.15 Hydraulic parameters estimated using the MC-E method for a selected event in each reach

Reach	Upstream Weir	Downstream Weir	Reach length (km)	V_{av} (m.s ⁻¹)	V_w (m.s ⁻¹)	R (m)	y (m)	S (%)	n
I	V1H038	V1H051	4	2.25	2.75	1.33	2.00	0.70	0.045
II	V2H002	V2H004	54	1.34	1.64	0.81	1.21	0.55	0.05
III	V2H004	V2H001	21	0.88	1.07	1.14	1.71	0.12	0.04

Table 3.16 Hydraulic parameters estimated using the MC-X method for a selected event in each reach

Reach	Upstream Weir	Downstream Weir	Reach length (km)	V_{av} (m.s ⁻¹)	V_w (m.s ⁻¹)	R (m)	y (m)	S (%)	n
I	V1H038	V1H051	4	1.82	2.22	0.97	1.45	0.70	0.045
II	V2H002	V2H004	54	1.51	1.85	0.97	1.45	0.55	0.05
III	V2H004	V2H001	21	0.88	1.07	1.13	1.70	0.12	0.04

3.6.4. Results

The results of applying the methodologies outlined in Section 3.6.2 are contained in this section. The results from only one event in each reach are reported. Additional results are reported by Tewolde (2005). The parameters for a selected event in each reach computed using the MC-E and MC-X methodologies are contained in Table 3.17 and Table 3.18 respectively.

Table 3.17 Estimated parameters using the MC-E method for a selected event

Reach	ΔL (m)	Δt (s)	A (m ²)	W (m)	Q_0 (m ³ .s ⁻¹)	K (s)	X	C_0	C_1	C_2
I	4090	1800	29.49	22.12	66.43	1486	0.48	0.97	0.11	-0.08
II	7777	5400	19.44	24.04	26.04	4743	0.49	0.99	0.07	-0.06
III	10000	9000	25.15	22.10	22.09	9317	0.46	0.92	0.02	0.05

Table 3.18 Estimated parameters using the MC-X method for a selected event

Reach	ΔL (m)	Δt (s)	A (m ²)	W (m)	Q_0 (m ³ .s ⁻¹)	K (s)	X	C_0	C_1	C_2
I	4090	1800	36.54	40.60	66.43	1841	0.49	0.97	0.00	0.02
II	7777	5400	21.03	21.75	26.04	4213	0.49	0.99	0.13	-0.12
III	10000	9000	25.22	17.00	22.09	9343	0.45	0.90	0.03	0.07

The computed and observed hydrographs from the applications of the MC-E and MC-X methods for the selected event in the three reaches are shown as Figure 3.42, Figure 3.43 and Figure 3.44 and the corresponding performance statistics are summarised in Table 3.19 and Table 3.20.

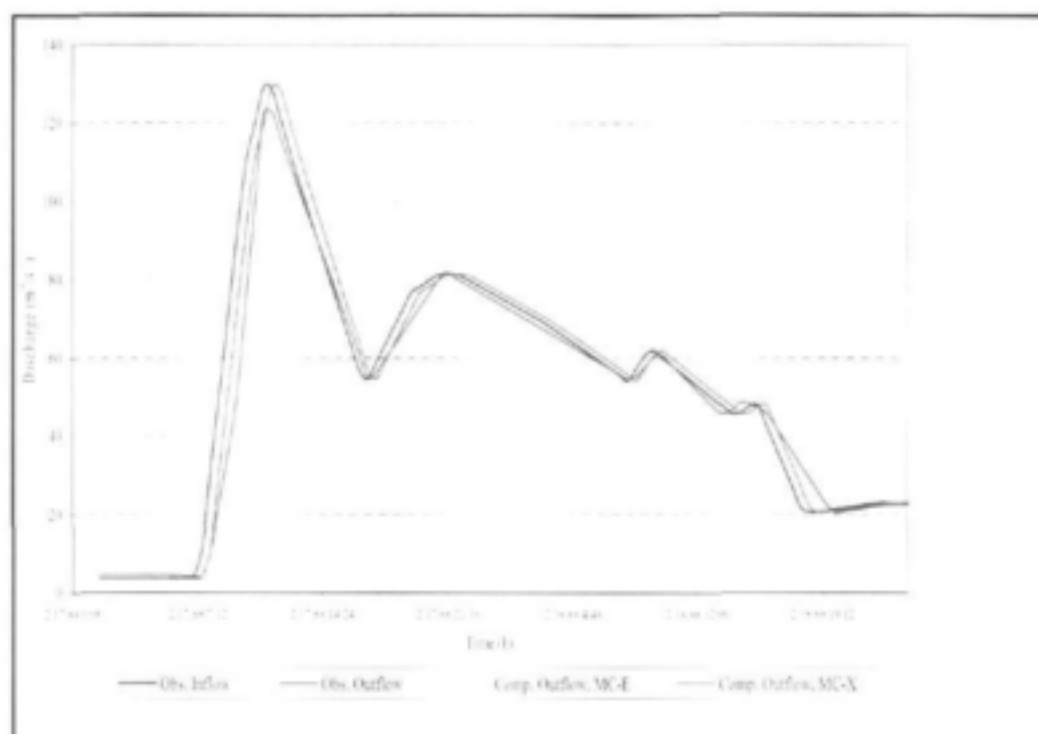


Figure 3.42 Observed and computed hydrographs in Reach-I

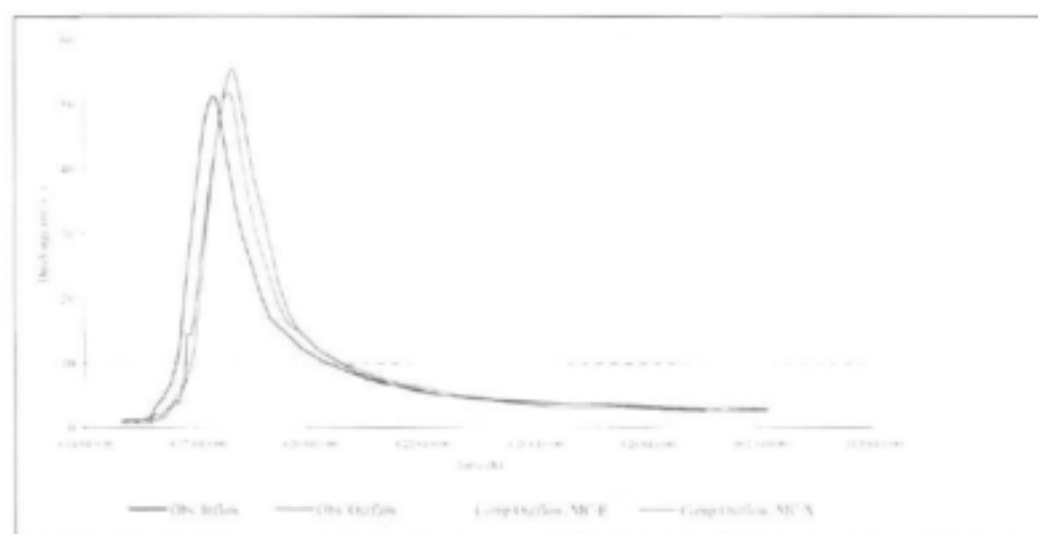


Figure 3.43 Observed and computed hydrographs in Reach-II

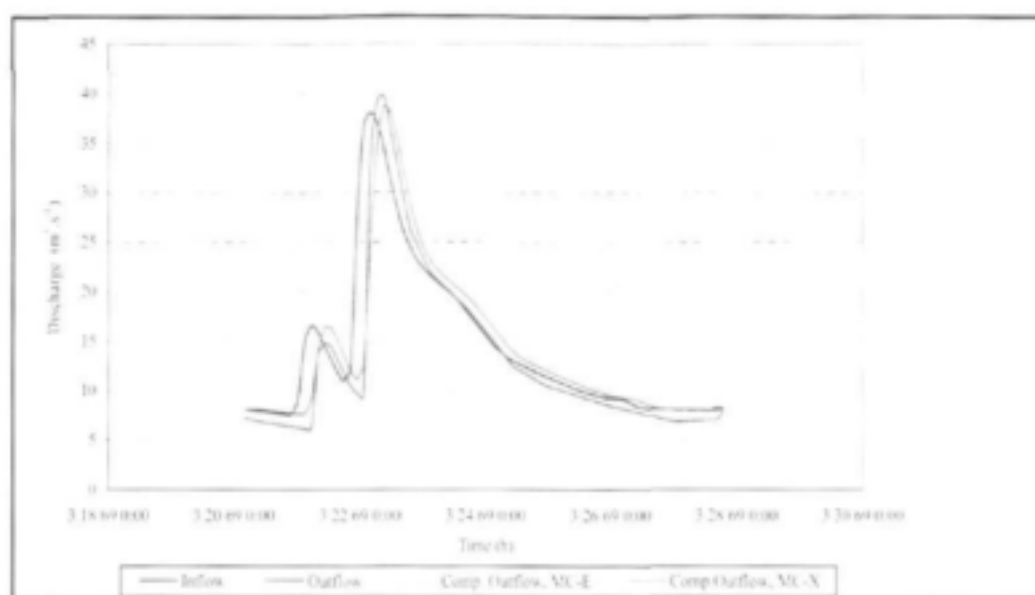


Figure 3.44 Observed and computed hydrographs in Reach-III

Table 3.19 Performance using the MC-E method of parameter estimation

Reach	Observed Peak Outflow ($\text{m}^3.\text{s}^{-1}$)	Comp Peak outflow ($\text{m}^3.\text{s}^{-1}$)	Peak Flow Error (%)	Peak timing Error (%)	RMSE ($\text{m}^3.\text{s}^{-1}$)	E	Obs Volume ($\text{m}^3 \times 10^6$)	Comp Volume ($\text{m}^3 \times 10^6$)	Volume Error (%)
I	122.79	129.77	5.68	-5.00	4.27	0.98	10.72	11.01	2.64
II	55.40	51.77	-6.55	0.00	2.02	0.97	14.39	13.76	-4.33
III	39.64	38.74	-2.27	0.00	1.77	0.95	8.72	9.45	8.37

Table 3.20 Performance using the MC-X method of parameter estimation

Reach	Observed Peak Outflow ($\text{m}^3.\text{s}^{-1}$)	Comp Peak outflow ($\text{m}^3.\text{s}^{-1}$)	Peak Flow Error (%)	Peak timing Error (%)	RMSE ($\text{m}^3.\text{s}^{-1}$)	E	Obs Volume ($\text{m}^3 \times 10^6$)	Comp Volume ($\text{m}^3 \times 10^6$)	Volume Error (%)
I	122.79	129.03	5.08	-5.00	3.92	0.98	10.72	11.00	2.61
II	55.40	51.70	-6.66	0.00	2.07	0.97	14.39	13.77	-4.28
III	39.64	38.53	-2.80	0.00	1.78	0.95	8.72	9.45	8.37

As shown in Table 3.19 and Table 3.20, the event in Reach II has a relatively small RMSE error and volume error with the coefficient of efficiency (E) nearly equal to one. The other events generally have acceptable statistical results. These results indicate a high degree of correlation between the computed and observed hydrographs. The result obtained from all three events show that the computed hydrographs are similar to the observed hydrographs, with errors of less than 10% for the statistics considered. Similar results were obtained by Tewolde (2005) for other events from the three reaches.

The addition of lateral inflow in the simulated hydrographs was not adequate when compared to the observed outflow in catchments where significant lateral flow was noted. As the length of a reach increases the possibilities of tributary inflows also increases. Hence, in large catchments it is recommended that the tributary flow should be added separately to the outflow hydrograph.

3.6.5. Discussion and Conclusions

Hydrological flood routing techniques are widely accepted and are extensively used in engineering practice. The ability to predict the changing magnitude and celerity of a flood wave as it propagates down rivers or through reservoirs makes flood routing important when designing hydraulic structure and in assessing the adequacy of measures for flood protection. However, in practice, only a limited number of flow gauging stations are available, and even measured or gauged runoff data are frequently unreliable. To establish gauging stations is an expensive task and on-going maintenance and service costs are also significant.

From the results obtained, it was generally found that the estimated values of the flow variables in the MC-E and MC-X methods resulted in computed hydrographs with acceptable errors for the peak flow magnitude, peak timing, volume and have small RMSE values. In addition, the coefficients of model efficiency (E) were generally close to 1 in most cases, which indicates that the simulated hydrograph shapes are very similar to the observed hydrographs. Hence, it can be concluded that both the MC-E and MC-X approaches can be applied in ungauged reaches in the Thukela Catchment where observed data sets are unavailable and it is postulated that the methodology would result in similar performance in other catchments in South Africa. The MC-E and MC-X method also has application in hydrological modelling where inflow hydrographs are simulated by a model and the hydrographs can be simulated at a down stream location using either the MC-E or MC-X methodology of parameter estimation.

3.7. Application of a Climate Classification System for Regional Adjustments of SCS Curve Numbers

As shown in Chapter 2, the SCS method is a design event based approach to design flood estimation and has been adapted for use in South Africa as reported by Schmidt and Schulze (1987). The refinements to the SCS method for South Africa include the development of techniques to account for typical soil moisture status prior to large storms.

In the SCS based methods, stormflow response is found to be highly sensitive to the catchment's wetness. Many researchers have proposed that procedures to adjust Curve Numbers (CNs) according to a soil water budget provide more realistic estimates of stormflow and peak discharge than when using only accumulated antecedent rainfall depths. However, the direct application of soil water budgeting procedures requires long and accurate daily rainfall and evaporation records, in order to estimate the change in soil moisture storage, ΔS , prior to runoff producing storm events. The problem then arises of how to estimate the values of change in soil water storage (ΔS) in regions where only limited hydrological and very basic monthly climatological information are available.

The soil water budget at a particular place or over a geographic area consists of daily moisture inflows, outflows and changes in storages. Most of the factors that affect the

soil water budget, viz. precipitation, evapotranspiration, moisture storage in the soil, surface runoff and the movement of water through the root zone within the soil profile are either implicit climatic parameters or are related to them explicitly. The aim of this component of the study was to estimate the soil water status of a catchment as a climatologically driven variable based upon the following hypotheses:

- In the absence of detailed daily water budget modelling to derive ΔS (owing largely to data constraints), ΔS will have to be derived/estimated by surrogate means.
- Ideally ΔS should be estimated from a readily available climatic surrogate, the simplest of which is MAP.
- Because a ΔS : MAP relationship is influenced by rainfall seasonality, rainfall concentration and seasonally variable evaporative demand, it is expected that the relationship may differ in different climatic zones.
- The Köppen climate classification (Köppen, 1931; cited by Blair, 1951) is a simple, generic, internationally accepted classification of climatic zones. Köppen climate classes (KCCs) have therefore been utilised to determine regional ΔS : MAP relationships.

ΔS is, furthermore, altered by *in loco* soil and vegetation characteristics. A distinction is therefore made, within a climate zone, between different ΔS : MAP relationships for major soil/vegetation combinations. These combinations should be identifiable from simple observations in the field without specialist knowledge. The combinations are made up of 3 major soil depth classes, 3 major soil texture classes and 3 biomass classes, as shown Table 3.21.

If the approach is shown to be successful, it is hypothesised that values of ΔS could be transferred to other regions with limited hydrological information, such as Eritrea, where such climate systems are known. The work reported in this section is a summary of the study conducted by Ghile (2004) which includes a detailed review of the relevant literature.

3.7.1. Methodology

In southern Africa, Schmidt and Schulze (1987) analysed change in soil water storage (ΔS) for large storms from the so-called "initial" catchment response conditions based on land use and soils characteristics. The analysis required the disaggregation of southern Africa into zones of relatively homogeneous hydrological response with regard to potential moisture recharge (rainfall) and atmospheric water demand (potential evaporation), based upon long term patterns of rainfall and temperature distributions. In total, 712 such zones were delimited by Dent *et al.* (1987).

Long term (~30 years) daily rainfall and temperature records were used in each of the 712 zones with the *ACRU* daily soil water budgeting model, which was run for each of the 3 soil texture, 3 soil depth and 3 land cover combinations (Table 3.21) to compute a soil moisture status prior to each stormflow producing rainfall event. This soil moisture status was compared to an initial value of S derived from an initial catchment Curve Number and the difference was designated ΔS . A frequency analysis of ΔS was undertaken to provide a direct estimate of ΔS for each zone and for each of the 27 soil depth, soil texture and land cover combinations for use in Equation 3.38, used for CN adjustment (Hawkins, 1978).

Table 3.21 Soil and vegetation characteristics used in soil moisture budget analysis (after Schmidt and Schulze, 1987)

Soil Thickness Classes (m)			
Category	Thickness of A Horizon		Thickness of B Horizon
Deep	0.30		0.80
Intermediate	0.25		0.50
Shallow	0.15		0.15
Moisture Retention Classes (mm/m)			
Category	Porosity	Drained Upper Limit	Lower Limit
Coarse (Sand)	430	112	50
Medium (Loam)	464	251	128
Fine (Clay)	482	416	298
Vegetation Classes			
Category	Fraction of Roots in A-horizon	Interception Loss (mm.rainday ⁻¹)	Crop Coefficient
Dense	0.60	3.00	1.00
Intermediate	0.80	1.75	0.75
Sparse	1.00	0.50	0.50

$$CN_f = \frac{(1+c) \cdot 1000}{\frac{(1+c) \cdot 1000}{CN_i} - \frac{(P_a - Q_a - D_a - E_a)}{25.4}} = \frac{(1+c) \cdot 1000}{\frac{(1+c) \cdot 1000}{CN_i} - \frac{\Delta S}{25.4}} \quad \dots \text{Eq. 3.38}$$

where

CN_f = final Curve Number calculated for prevailing soil moisture status at the end of the defined interim period,

CN_i = initial Curve Number for the hydrological soil-cover complex of the catchment,

c = coefficient of initial abstraction,

P_a = rainfall (mm) in the interim period,

Q_a = stormflow (mm) in the interim period,

D_a = drainage (mm) in the interim period,

E_a = actual evapotranspiration (mm) in the interim period, and

ΔS = change in soil water status (mm) in the interim period, i.e.
 $= P_a - Q_a - D_a - E_a$.

The results are documented in the appendices of Schmidt and Schulze (1987), and information from these appendices are the basic references for this study. The major tasks undertaken by Ghile (2004) included the following:

- a literature review,
- the selection of a climate classification system (Köppen system),
- the creation of Köppen climate classification (KCC) for South Africa and data sparse region (Eritrea),
- analysis of the 712 relatively homogeneous climate zones in South Africa according to the KCC to check if similar KCC have similar CN adjustment, and
- verification of the SCS model with climatically driven CN adjustment, and ACRU models on an research catchment in Eritrea (Afdeyu).

Based on the review of climate systems by Ghile (2004), the Köppen climate classification system (Köppen, 1931; cited by Blair, 1951) was selected for this study because of its relative simplicity when compared to the Thornthwaite (Thornthwaite and Mather, 1955) and FAO (FAO, 2002) agro-ecological climate classification systems, as well as the availability of spatially detailed monthly and annual rainfall and temperature information in Eritrea.

The first hypothesis was tested by applying the Köppen climate classification to the 712 relatively homogeneous hydrological zones which have been identified in southern Africa. The Köppen (1931) climate classification system uses values of long term monthly means of daily temperature and long term mean monthly and annual precipitation to identify five major climatic classes, A to E. Each climate class contains sub-classes which describe more detailed regional climate characteristics, such as seasonal changes in temperature and precipitation. Figure 3.45 shows the distribution of Köppen climates overlayed over the relatively homogeneous 712 climatic zones in southern Africa identified by Dent *et al.* (1987). For this research, only those homogeneous zones that fell entirely within the same KCC were used for further analysis. This resulted in 550 of the 712 zones (77%) being used in this study. Moreover, the KCCs Am, Aw, Cwc and ET were excluded from further analysis as no homogeneous zone within southern Africa fell entirely in these KCCs.

Assuming for design hydrology at a specified location that a statistically expected, or median (50th percentile), value of ΔS could be used in Equation 3.38 to derive final Curve Numbers, values of ΔS_{50} , obtained previously by Schmidt and Schulze (1987) for 3 of the 27 soil/vegetation cover combinations, were then analysed to check if zones having similar Köppen climate classes (KCCs) have similar values of ΔS . The 3 soil/vegetation cover scenarios were, sparse vegetation on shallow clay soils (SCSV), intermediate vegetation on intermediate depth loamy soils (ILIV), and dense vegetation on deep sandy soils (DSDV).

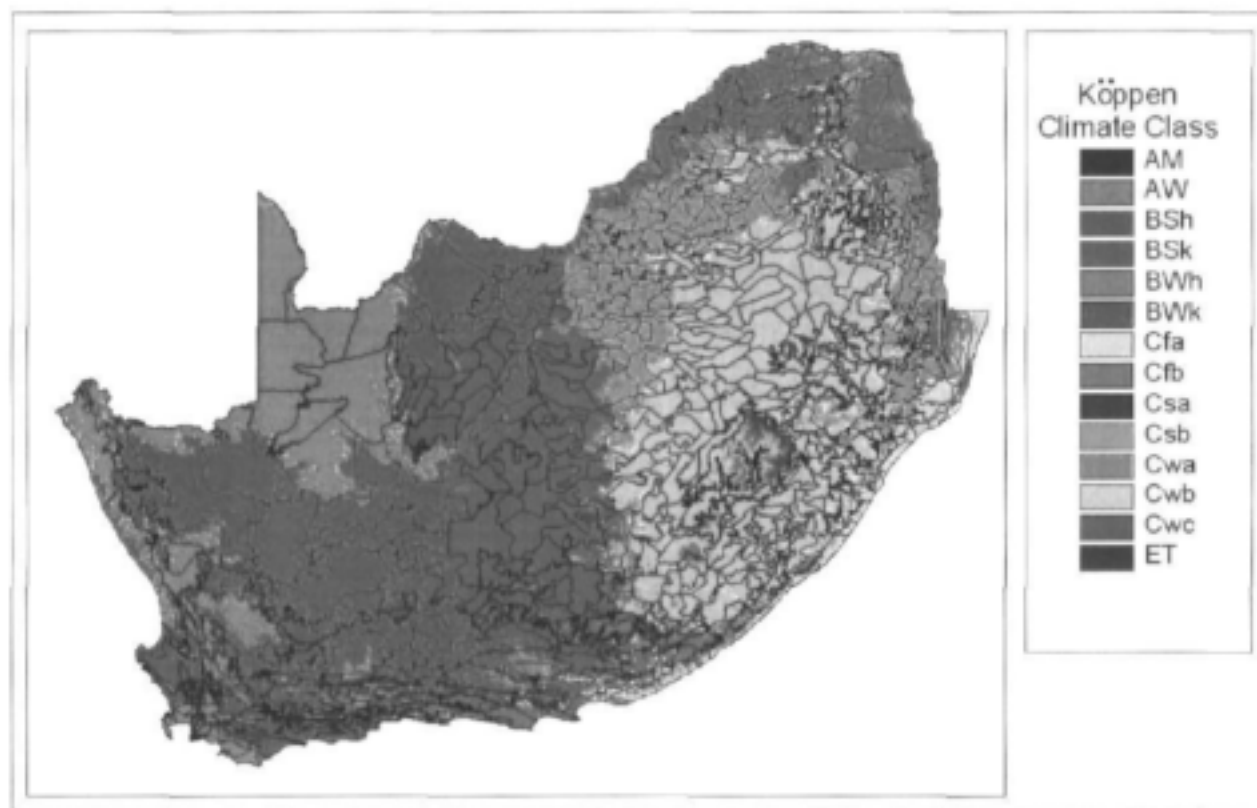


Figure 3.45 Köppen climate classes overlaid over the 712 relatively homogeneous hydrological zones of southern Africa

3.7.2. Results

In the analysis of variances within each KCC identified in southern Africa, generally a high degree of homogeneity in values of ΔS_{50} was observed for the selected three soil/vegetation cover combination scenarios. Further analysis showed that the values of ΔS_{50} are well explained by the linear regression of MAP in nearly all the KCCs identified in southern Africa. However, in the *Csb* climate class the values of ΔS are more strongly explained by non-linear (logarithmic) relationships, even though the linear relationship is still statistically significant (Figure 3.46 to Figure 3.48).

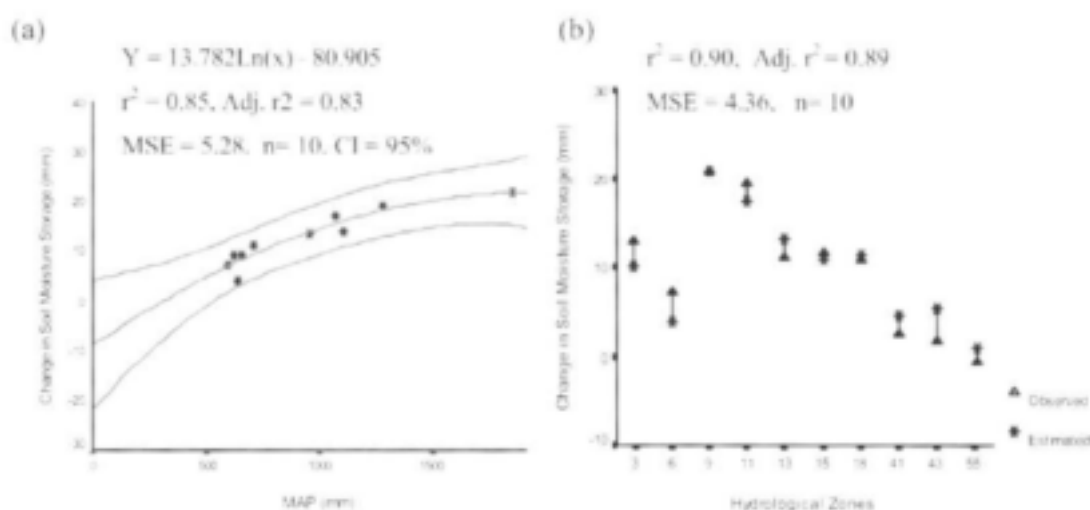


Figure 3.46 (a) Scatter diagram of median antecedent storage changes, ΔS , vs MAP and (b) simulated vs observed ΔS values for those hydrologically relatively homogeneous zones in southern Africa with a *Csb* climate, for catchments assuming sparse vegetation on shallow clay soils (SCSV)

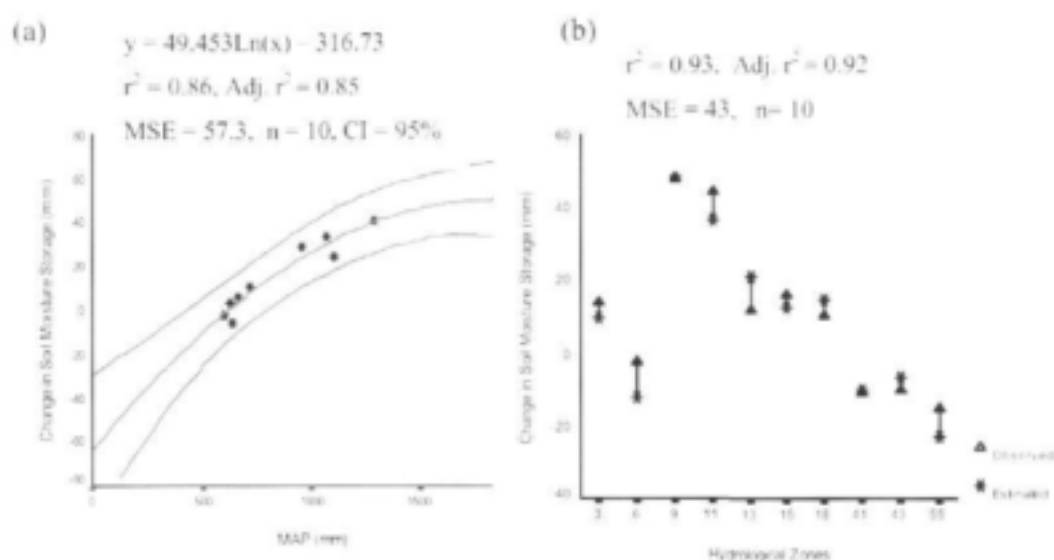


Figure 3.47 (a) Scatter diagram of median antecedent storage changes, ΔS , vs MAP and (b) simulated vs observed ΔS values for those hydrologically relatively homogeneous zones in southern Africa with a *Csb* climate, for catchments assuming intermediate vegetation on intermediate loamy soils (ILIV)

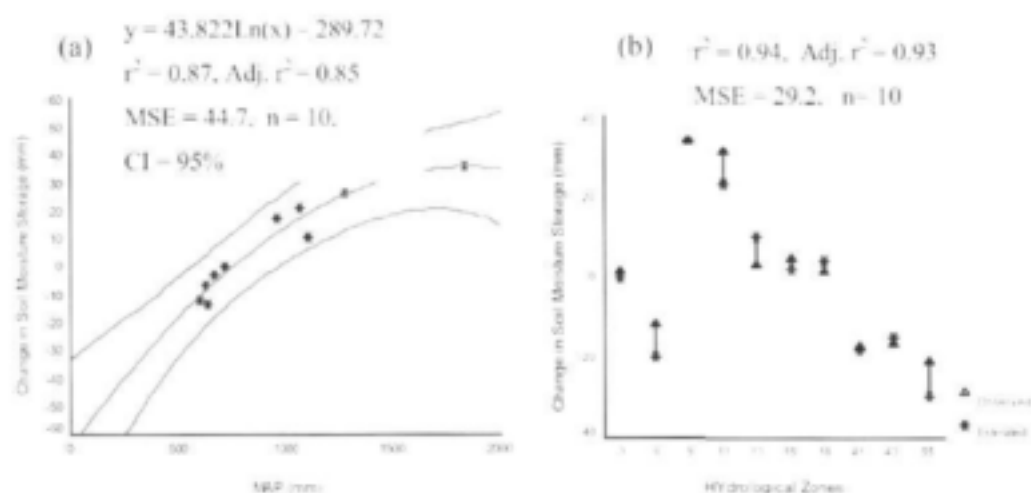


Figure 3.48 (a) Scatter diagram of median antecedent storage changes, ΔS , vs MAP and (b) simulated vs observed ΔS values for those hydrologically relatively homogeneous zones in southern Africa with a *Csb* climate, for catchments assuming dense vegetation on deep sandy soils (DSDV)

In Figure 3.49, Figure 3.50 and Figure 3.51 the relationships between ΔS_{50} and MAP are illustrated for all the Köppen climate classes identified in southern Africa for the three soil and land cover scenarios. As may be seen in Figure 3.49 for the SCSV scenario, a single trend line can explain the relationships between the ΔS and MAP for

all the KCCs identified in southern Africa, except for the *Csb* class. Such a small variation in ΔS might be attributed to the relatively insignificant effect that a highly responsive shallow clay soil with sparse vegetation cover would play in the variation of ΔS , because most of a day's rainfall would be converted to runoff on the same day the rainfall event occurs.

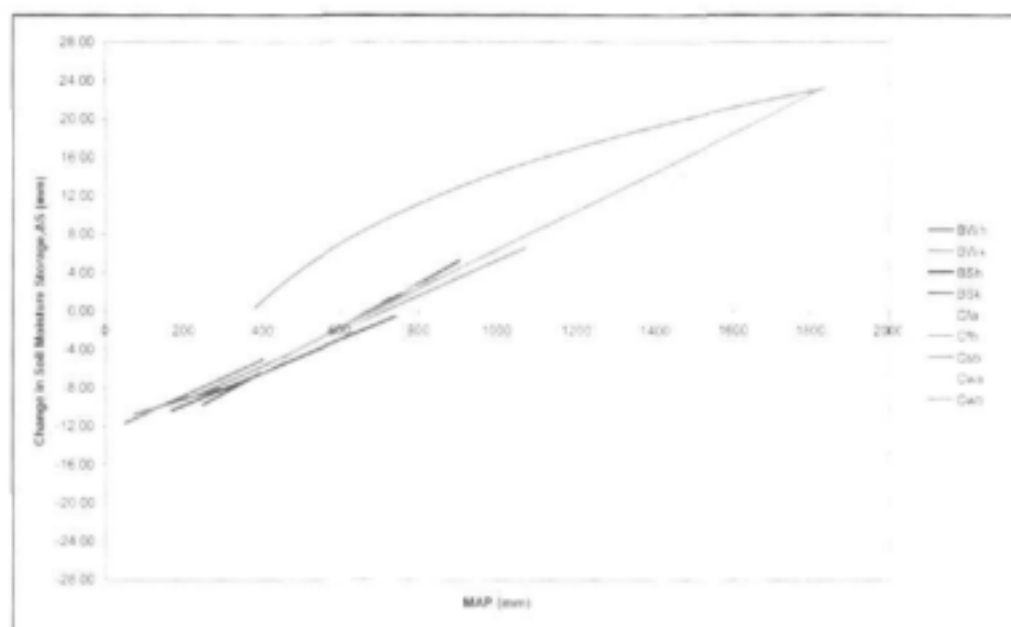


Figure 3.49 Relationships between median antecedent soil storage changes, ΔS_{50} , and MAP for the Köppen climate classes identified in southern Africa, when catchments are covered with sparse vegetation on shallow clay soils, SCSV

The ΔS : MAP relationship is, however, seen to vary markedly for KCCs for the ILIV and DSDV scenarios (Figure 3.50 and Figure 3.51), where the interplay between different soil/land use combinations and prevailing climatic conditions becomes important.

The high correlation between estimated and observed ΔS_{50} values within each KCC is also an indication of a good simulation of ΔS_{50} from MAP by the regression equations which are unique to each KCC (Figure 3.46 to Figure 3.48). The conclusions reached in the analysis of ΔS_{50} values within each KCC may be summarised as follows:

- A high degree of homogeneity of ΔS_{50} was observed within each individual KCC identified in southern Africa.
- The values of ΔS_{50} , which were previously computed with the *ACRU* model by Schmidt and Schulze (1987) are strongly correlated with MAP within each of the KCCs identified in southern Africa.
- The Köppen climate classification system can be used as a surrogate method for the adjustment of Curve Numbers for antecedent soil moisture status, and KCCs can be delineated when only very basic mean monthly climatological information is available.

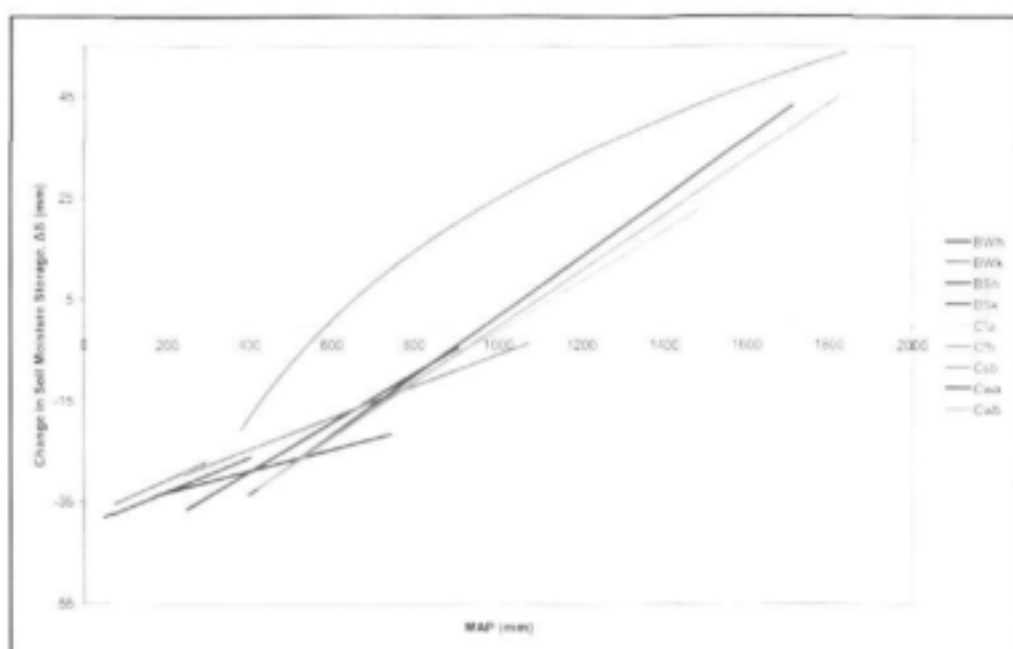


Figure 3.50 Relationships between median antecedent soil storage changes, ΔS_{50} , and MAP for the 9 Köppen climate classes identified in southern Africa, when catchments are covered with intermediate vegetation on intermediate depth loamy soils, ILIV

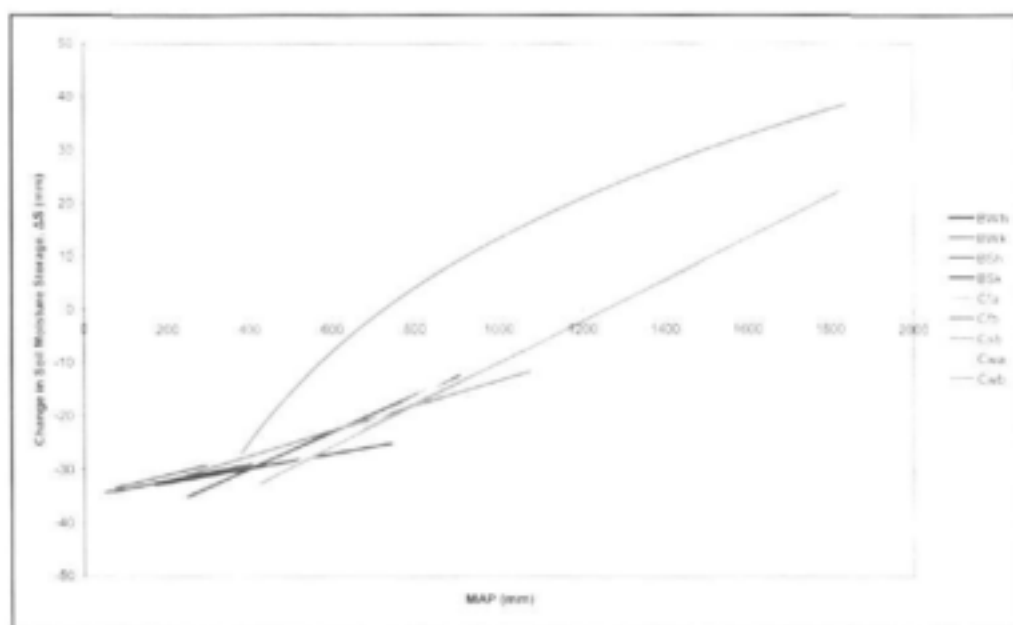


Figure 3.51 Relationships between median antecedent soil storage changes, ΔS_{50} , and MAP for the Köppen climate classes identified in southern Africa, when catchments are covered with dense vegetation on deep sandy soils, DSDV

The next task undertaken was to verify whether the typical regionalised indices of ΔS computed with the *ACRU* model by Schmidt and Schulze (1987) for a wide spectrum of climatic conditions, soil properties and land use characteristics could be transferred to Eritrea by application of the Köppen climate classification.

The performance of the *ACRU* stormflow modelling approach was first tested on the Afdeyu research catchment (1.77 km^2) in Eritrea, in order to be confident in the use of values of ΔS generated by the model. Despite the relatively limited level of information on climate, soils and land use for the Afdeyu catchment, the monthly totals of simulated daily flows from the *ACRU* mimicked the corresponding observed flows excellently as shown in Figure 3.52 with the one exception of August 1998, where the model overestimated by approximately 28%.

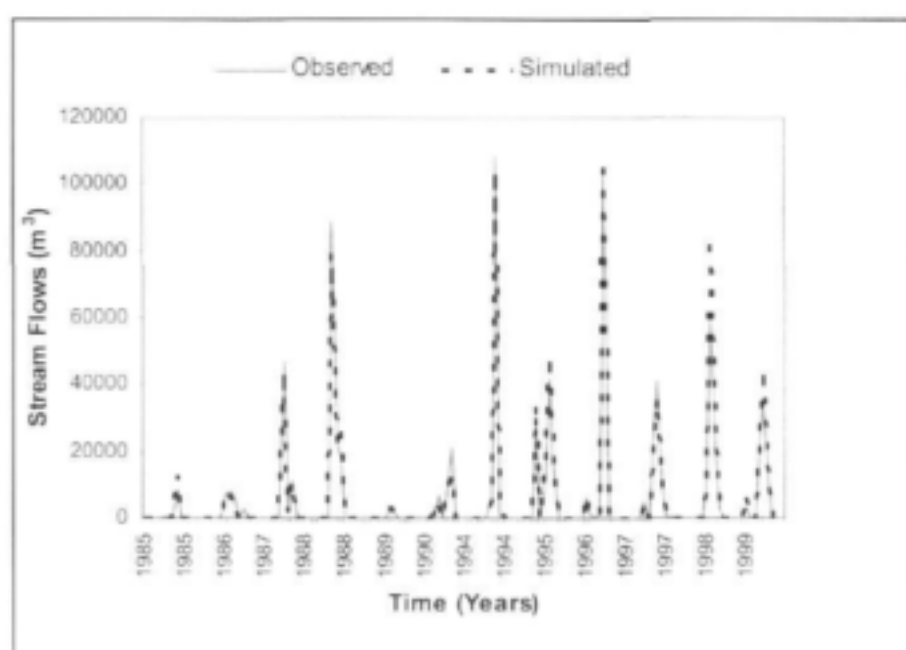


Figure 3.52 Monthly totals of daily *ACRU* simulated versus observed flows (m^3) in the Afdeyu catchment for the period of 1985-1999, with missing observed data from 1991-1993

A plot of simulated versus observed daily flows, as well as monthly totals of daily flows, reveals a very high performance of the model, with $r^2 = 0.94$ for the analysis of daily flows (Figure 3.53a) and $r^2 = 0.97$ for the analysis of monthly totals of daily flows (Figure 3.53b). The relationship is especially strong for less extreme events, while there is slightly more scatter at high flows. While there might have been occasional misreading of the raingauge, or wind flow causing turbulent eddies around the raingauge orifice, which occurs commonly in the torrential and erratic storm events experienced at Afdeyu, only one outlier point was found in the daily analysis.

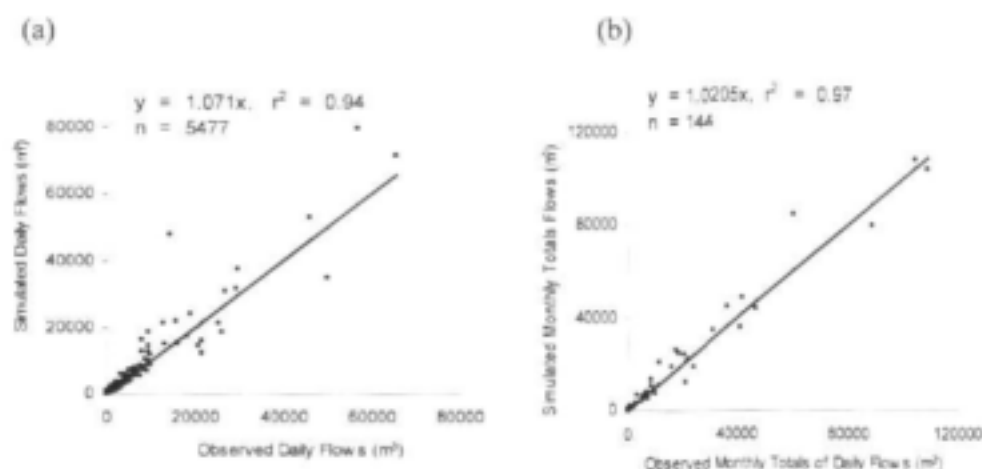


Figure 3.53 (a) Scattergrams of *ACRU* simulated daily flows and (b) monthly totals versus observed flows for the Afdeyu catchment for the period 1985-1999, with missing data from 1991-1993

The excellent simulations of the *ACRU* model are confirmed by the accumulated monthly and annual flows, as shown in Figure 3.53, which illustrate clearly that the total values of flows were well simulated. The above results, although from only one research catchment in Eritrea, illustrate that the *ACRU* model may be used with confidence in further studies.

The concept of using the *ACRU* model to derive ΔS_{50} and then relating the ΔS_{50} to MAP for a given KCC was termed the *ACRU*-Köppen approach. The *ACRU*-Köppen method was tested on the same catchment and results were compared to stormflows using the SCS-AMC classes and Hawkins' soil water budgeting procedures for CN adjustment. In a comparison of design stormflows estimated using the SCS-SA model, those CNs adjusted by the water budget and by the surrogate *ACRU*-Köppen method provided results in close agreement with the observed design stormflow volumes when compared to the stormflow estimates obtained by SCS-AMC classes and unadjusted CN methods. Based on a ranking of results for the four selected return periods, the *ACRU*-Köppen slightly outperformed the water budget technique (Table 3.22). The model simulation by the original SCS-AMC classes was very poor, with stormflow volumes for 2 and 5 return year periods under-estimated by approximately 65% and 28% respectively, while for the 20 year return period it was over-estimated by 53%. Using the so-called "initial" catchment soil moisture conditions (CN unadjusted), the model generated realistic flow estimates for 2 and 5 return year periods, but under-estimated by around 10% and 15% for the 10 and 20 year return periods respectively (Figure 3.54).

Table 3.22 Design stormflows for selected return periods at the Afdeyu catchment, simulated by four SCS-based techniques and with ranking given in parenthesis

Return Period (years)	Observed Flows (m ³)	Design Flows (m ³)			
		CN Adjusted by Water Budget	CN Adjusted by <i>ACRU</i> -Köppen	CN Unadjusted	CN Adjusted by SCS-AMC Classes
2	18 074	20 723 (1)	20 947 (2)	22 380 (3)	6 186 (4)
5	41 097	45 390 (3)	45 390 (3)	41 444 (1)	29 637 (4)
10	59 415	64 007 (3)	61 978 (1)	53 725 (4)	63 612 (2)
20	78 957	81 068 (2)	77 659 (1)	67 418 (3)	121 399 (4)

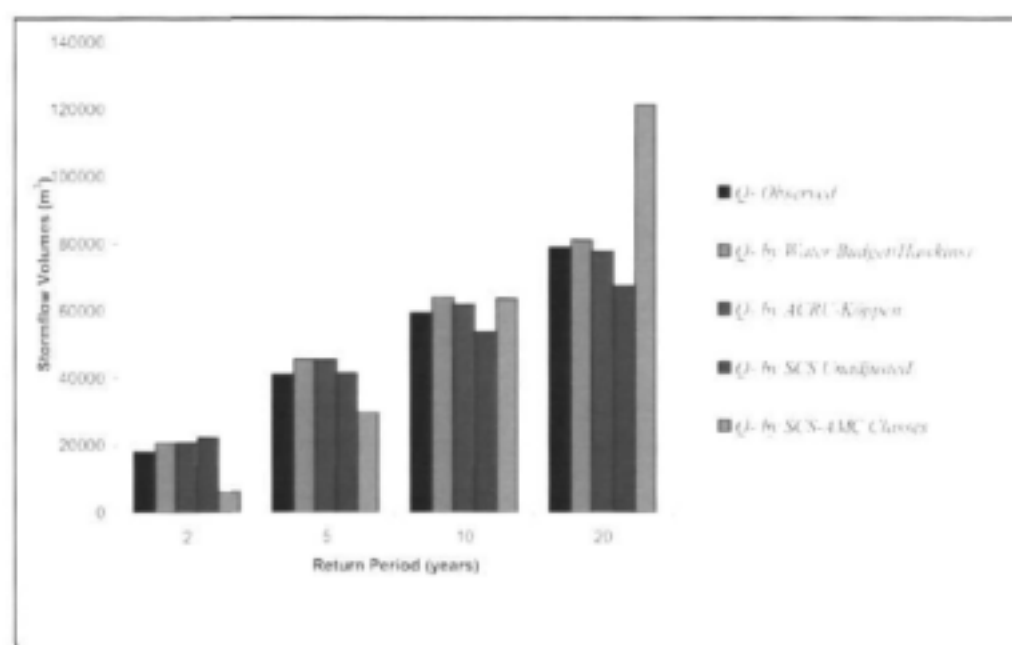


Figure 3.54 Design stormflow volumes for the Afdeyu research catchment generated from design daily rainfalls using water budget, *ACRU*-Köppen, SCS unadjusted and SCS-AMC methods of Curve Number adjustments

Regression equations developed by Ghile (2004) to derive AS_{50} from MAP within each individual KCC identified in southern Africa, for a combination of the three soil depth categories, three soil texture classes and three vegetation cover conditions, are listed in the SCS-LHI user manual (Schulze and Ghile, 2004) and it is hypothesised that those equations can be used anywhere for the same KCCs and similar soil/land cover characteristics.

The outputs from both the *ACRU* and SCS-based models for the five highest stormflows produced from the five highest daily rainfall amounts of each year in the Afdeyu catchment were compared statistically Table 3.23. The *ACRU* model provided excellent overall simulated stormflow volumes and had the lowest difference in

standard deviations between observed and estimated flows. The coefficient of determination was highly acceptable and the slope was much closer to unity than for either of the SCS-based estimates. The fact that the *ACRU* model generated consistently better stormflow volumes implied that the antecedent soil moisture component appears to have a marked effect on the accurate design flood estimation. In comparing results from the different methods of CN adjustment used in the SCS model, the *ACRU*-Köppen yielded much better model efficiencies than the two other methods. The coefficient of determination and coefficient of efficiency exhibited by the *ACRU*-Köppen are markedly higher than those of either of the two methods of CN adjustments.

Table 3.23 Statistics of performance of the *ACRU*, SCS adjusted by *ACRU*-Köppen, SCS adjusted by AMC classes and the unadjusted SCS models for the five highest stormflows produced from the five highest daily rainfall amounts of each year at the Afdeyu catchment

Conservation Statistics				
Statistics of Daily Flows	<i>ACRU</i>	CN Adjusted by <i>ACRU</i> -Köppen	CN Adjusted by SCS-AMC Classes	Unadjusted SCS
Total observed flows (m ³)	695 325.5	695 325.5	695 325.5	695 325.5
Total simulated flows (m ³)	714 503.5	750 466.7	569 017.2	754 995.0
Percentage difference in total flows	2.8	7.9	-18.2	8.6
Standard deviation of observed flows (m ³)	14 399.4	14 399.4	14 399.4	14 399.4
Standard deviation of simulated flows (m ³)	14 696.1	12 796.5	19 495.3	12 446.0
Difference in standard deviations (%)	2.1	-11.1	-35.4	13.6
Regression Statistics				
Statistics of Daily Flows	<i>ACRU</i>	CN Adjusted by <i>ACRU</i> -Köppen	CN Adjusted by SCS-AMC Classes	Unadjusted SCS
Correlation coefficient	0.96	0.92	0.65	0.83
Slope of the regression line	0.98	1.02	0.87	0.72
Base constant for regression equation	529.4	362.4	-651.1	4305.4
Coefficient of determination	0.92	0.85	0.42	0.69
Coefficient of efficiency	0.92	0.72	0.40	0.56

3.7.3. Discussion and Conclusions

The *ACRU*-Köppen approach is a conceptually sound, computationally simple, complies with hydrological fundamentals and does not violate the original intent of the SCS technique. A major strength of the approach is that it is a relatively simple technique, does not require a high level of expertise and is not data demanding for its application. In view of the above findings, the following conclusions were drawn regarding the suitability of the *ACRU*-Köppen method of CN adjustment for application in developing countries:

- In an Eritrean context it appears that the original CN adjustment which utilises only rainfall amounts and “average” catchment conditions to represent the antecedent soil moisture status does not provide a sound basis for design hydrology on small catchments.
- Using the *ACRU*-Köppen method of CN adjustment, the SCS-SA model can be adapted to Eritrea, for which Köppen climates can be classified from monthly rainfall and temperature maps. However, it remains essential to know whether a climate is relatively constant from one year to the next, or whether a high variation is experienced from one season to the next, since the delimitation of such climate zones is done empirically and can, therefore, vary from year to year (climate variability) or over time (climate change).
- With improved hydrological and climatological information, the *ACRU* model can also be adapted as a tool to simulate a range of other water related issues in Eritrea by testing the model further on a wide range of catchment characteristics.

It should be noted that the surrogate method of CN adjustment which was conceptualised and verified in this study, viz. the *ACRU*-Köppen approach, was tested only on one research catchment in Eritrea, as there are currently no other gauged catchments in Eritrea available for undertaking such verifications. In future, this method should be tested against the other methods of CN adjustments under a wider range of environmental conditions. As indicated in the literature review (Chapter 3, Section 3.2.2) documented by Ghile (2004), spatial differences in a catchment's soil moisture status are attributed to variations in one or more of the following: regional climate, soil characteristics, land use and its management and topographical position in the landscape. The following recommendations for future investigations are therefore suggested.

Further improvements may be possible by including the effects of topography and land management into the estimation of regional indices of antecedent soil moisture change, ΔS . Particularly in humid and sub-humid climates (e.g. *Cfa*, *Cfb*, and *Cwb*), where precipitation generally exceeds evapotranspiration, topography has a significant influence on spatial patterns of antecedent soil moisture change on a catchment, as the upslope topography can modify the distribution of ΔS at a given point downslope.

It should also be noted that in arid and semi-arid climates (e.g. *BWh*, *BWk*, *BSh* and *BSk*) most of the rainfall is recorded within a few months of the year only, and over very short time periods (convective events). Therefore, the duration and number of the storm incidences are crucial variables that should not be ignored in determining the soil water status of a given area. In other words, to what extent do spatial patterns of ΔS persist across time in arid and semi-arid areas? Temporal variations in rainfall have influences probably as strong as those of spatial variations in the soil water balance.

4. APPLICATION OF THE CONTINUOUS SIMULATION MODELLING SYSTEM: A CASE STUDY IN THE THUKELA CATCHMENT

The Thukela catchment (29 036 km²), located as shown in Figure 4.1, was selected as a case study for this project. The catchment is classified as a Primary catchment by the Department of Water Affairs and Forestry (DWAF) and is a designated Water Management Area which will evolve into a Catchment Management Agency (Schulze *et al.*, 2005a). The Thukela Catchment is diverse with a wide range of altitudes (0 - 3000+ m), varying annual rainfall (550 - 2 000 mm), and different soils and ecological regions. The catchment thus represents a range of hydrological regimes to assess the continuous simulation modelling system.

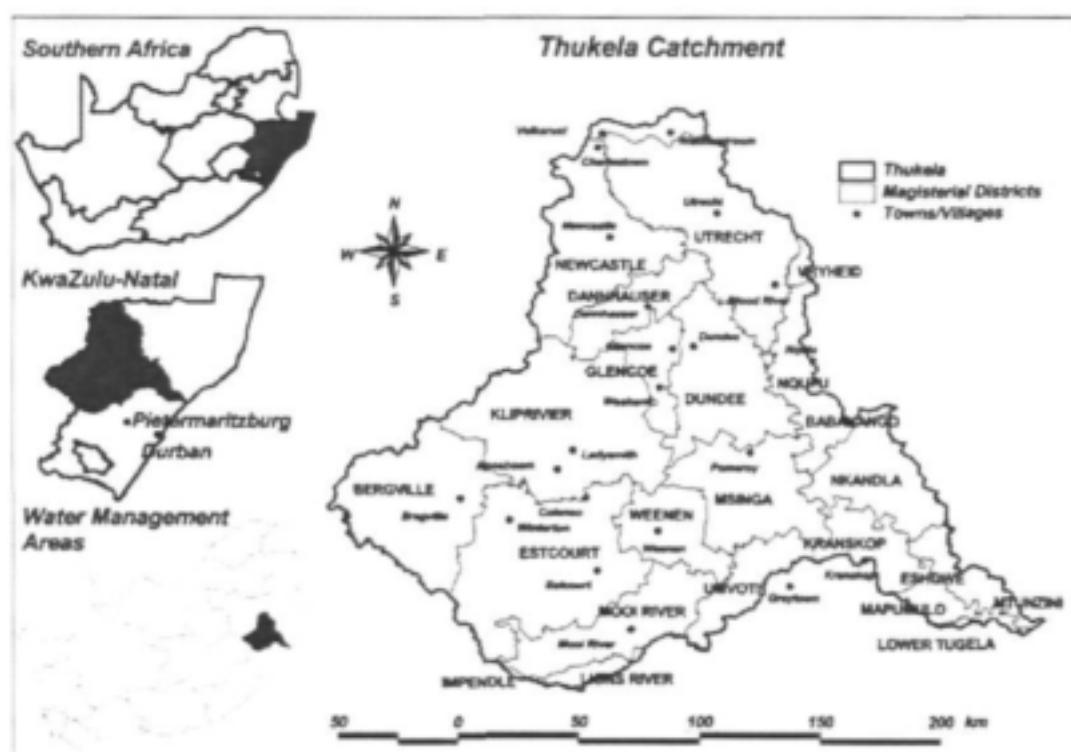


Figure 4.1 Location of the Thukela catchment in relation to KwaZulu-Natal province, designated Water Management Areas in South Africa, magisterial districts and major towns within the catchment (Schulze *et al.*, 2005a)

4.1. Configuration of the ACRU Model

At the fourth level of catchment delineation in South Africa, the Thukela catchment has been delineated into 86 Quaternary Catchments (QCs). As a consequence of the diverse hydrological response within many of the 86 QCs, Schulze *et al.* (2005a) further subdivided the 86 QCs into 235 sub-catchments based on natural heterogeneities (altitude, rainfall, altitude, temperature/evaporation), soils, natural vegetation, channel-based factors (location of streamflow gauging weirs and dams), environmental flow considerations (Environmental Flow Requirement sites) and political history (degraded vegetation). A comparison of the 86 QC and 235 sub-

catchment delineation in the Thukela Catchment is shown in Figure 4.2. A schematic representation of the linked 235 sub-catchments is shown in Figure 4.3.

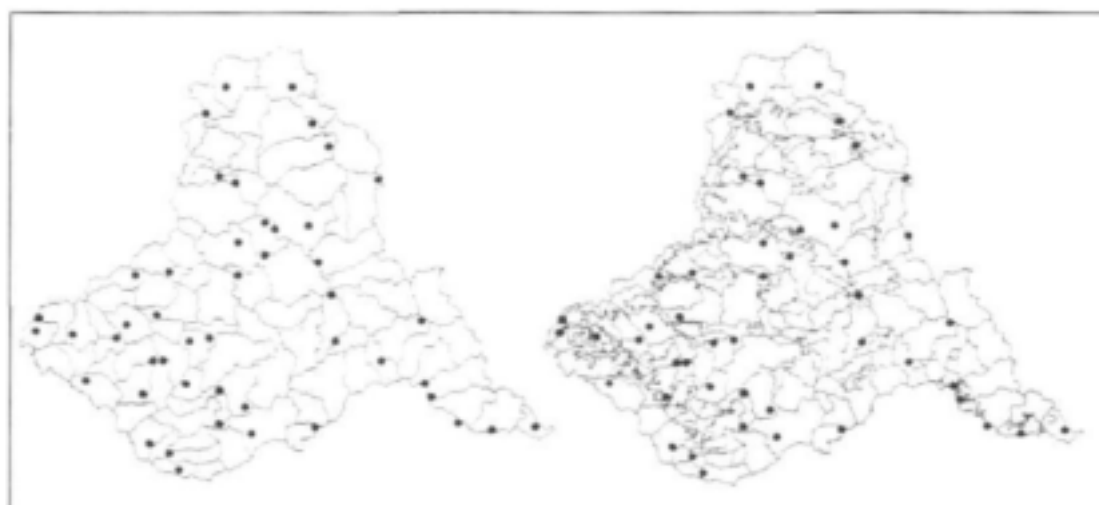


Figure 4.2 Comparison of the 86 Quaternaries in the Thukela Catchment (left) and the 235 sub-catchments (right), with dots representing the rainfall station network (Schulze *et al.*, 2005a)

The configuration of the *ACRU* model for the Thukela Catchment by Schulze *et al.* (2005b) was used as a basis for the configuration used in this study. As described by Schulze *et al.* (2005b) a single daily rainfall station was selected to represent each of the sub-catchments. The daily rainfall extraction utility developed by Kunz (2004) was used to extract the 10 closest rainfall stations to the centroid of each sub-catchment. These 10 stations were ranked by the Daily Rainfall Extraction Utility (Kunz, 2004) using criteria such as distance from the rainfall station to the centroid, operational period of record, differences between the MAP of the centroid and rainfall data and the reliability of the data, which is based on the fraction of record which is infilled. Schulze *et al.* (2005b) then manually selected an appropriate driver rainfall station for each catchment based on the reliability of the record, the ranking given by the Daily Rainfall Extraction Utility, the topography of the catchment, prevailing weather direction and, to a large extent, hydrological knowledge/experience of a team of evaluators.

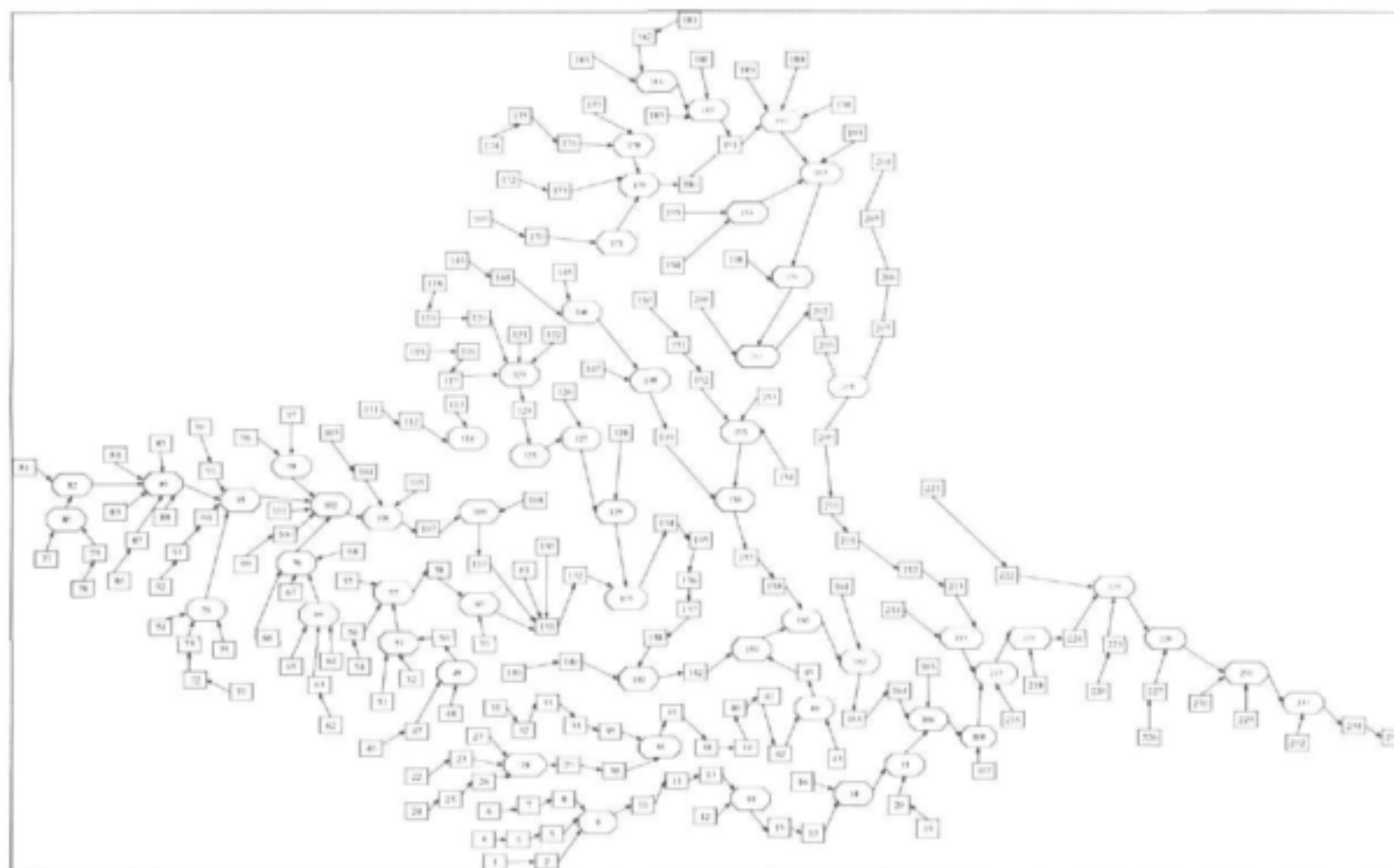


Figure 4.3 Configuration and sub-catchment numbering of 235 sub-catchments in the Thukela Catchment

The daily rainfall datasets were selected from quality controlled rainfall values collated by Lynch (2004) and daily maximum and minimum temperatures extracted from the database developed by Schulze and Maharaj (2004). As explained by Schulze *et al.* (2005a), the 1' x 1' gridded values of MAP developed from a Geographically Weighted Regression by Lynch (2004) were used to determine an area-weighted MAP for each of the 235 sub-catchments and 12 monthly weighting factors were assigned to the daily data from the most representative rainfall station for each sub-catchment. The period of simulation was from 1950 to 2000. The Hargreaves and Samani (1985) daily equation option in the *ACRU* model was used to estimate daily reference potential evaporation.

Soils information was obtained from the ISCW soils maps (SIRI, 1987) at the 1:50000 scale which has been translated into *ACRU* variables using a methodology developed by Pike and Schulze (1995). Land cover information was obtained from the National Land Cover Database (CSIR, 1999) at a resolution of 1: 250 000 and has been translated into *ACRU* variables by Schulze (2001).

The results presented in Chapter 3, Section 3.2, indicate that it is necessary to model Hydrological Response Units (HRUs) within sub-catchments to simulate the most realistic hydrological response. HRUs were thus identified within each of the 235 sub-catchments, based on the land cover within each sub-catchment. The HRUs are thus modelled as "sub-catchments" with no fixed spatial location within a sub-catchment. An example of the linkages of the HRUs used to form a physical sub-catchment is shown in Figure 4.4. The location of flow gauging weirs, identified by Joubert and Hurley (1994) as having good quality data and relatively un-impacted catchments, is shown in Figure 4.5.

4.2. Simulation of Irrigation

The area under irrigation, crop types and modes of irrigation scheduling influence the volume of runoff abstracted for irrigation and hence on the runoff simulated from a sub-catchment. Information in Midgley *et al.* (1994a), the National Land Cover Database (CSIR, 1999) and the Water Use Licencing, Registration and Revenue Collection (WARMS) database, which contains information on registered water users compiled by DWAF, were sources of information on irrigated areas in the Thukela Catchment. An extract from the WARMS database for the Thukela Catchment was obtained from Tylecoat (2005). A comparison of the irrigated areas at a quaternary catchment scale and the accumulated area per quaternary catchment used in the simulation is contained in Table 4.1. It was deemed that the WARMS database contained the most up-to-date and accurate information and hence the irrigated areas in the WARMS database were utilised in this study. Although the latitude and longitude contained in the WARMS database enable the apportionment of the irrigated area to a particular sub-catchment, it was found that the recorded locations of the irrigated areas were not accurate, with many registered irrigation areas appearing outside of the Thukela Catchment. Hence, the total irrigated area recorded for each quaternary catchment in the WARMS database was allocated to sub-catchments making up the quaternary catchment in the same proportion as the irrigated areas in the sub-catchments identified by the CSIR (1999) from satellite imagery.

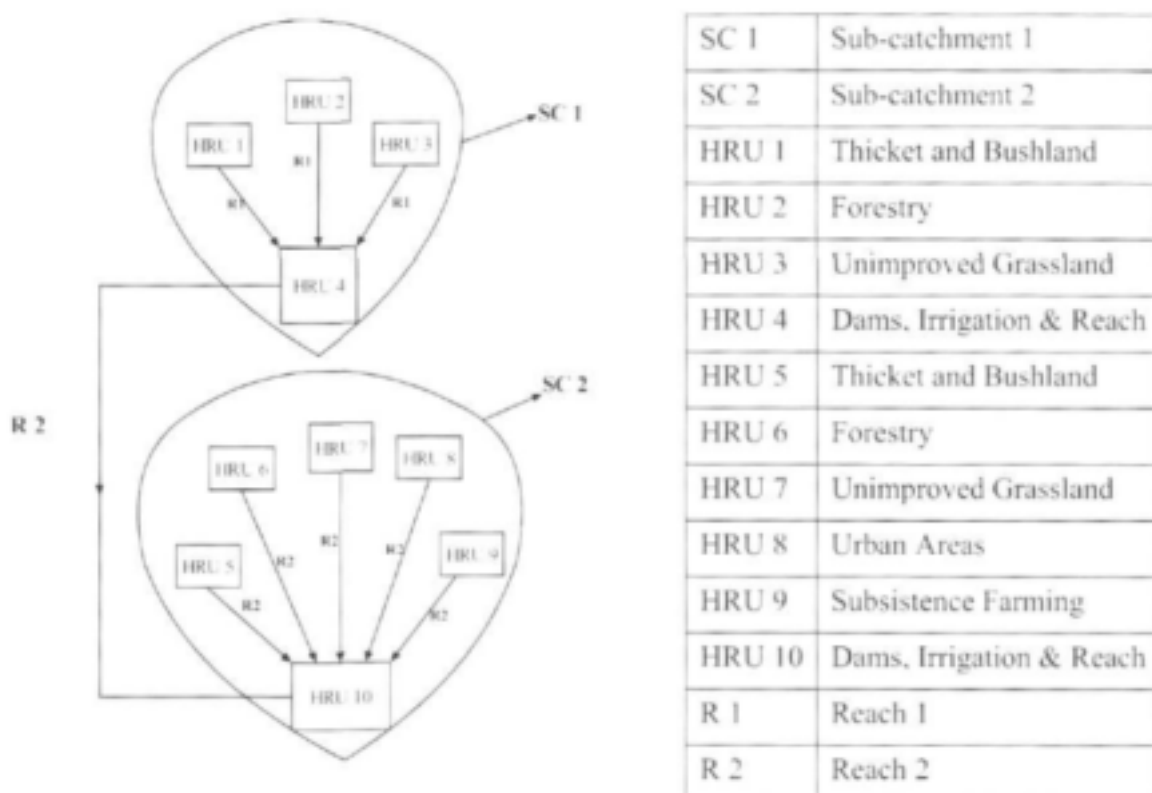


Figure 4.4 Schematic diagram showing an example of HRUs and reaches (R) modelled to form two sub-catchments (SC)



Figure 4.5 Raingauges and flow gauging weirs in the Thukela Catchment

Two irrigation scheduling modes were simulated. The first (ISCHE=1) is dependent on the climate and soil. Irrigation is initiated when the soil water in the root zone falls below 50% of the plant available water, at which point an amount is irrigated to refill the soil profile to a level (5 mm) below the drained upper limit of the soil, in order to utilise any rainfall which may occur shortly after an irrigation event. The second irrigation schedule simulated (ISCHE=2) was a fixed amount (15 mm) applied in a fixed cycle (7 days). The cycle is reset if 15 mm or more rainfall occurs.

Table 4.1 Irrigated areas in the Thukela Catchment

QC No.	Quaternary Catchment	Irrigated Area (km ²)		
		Midgley	CSIR (1999)	WARMS
1	V20A	3.80	8.73	2.69
2	V20B	4.20	23.23	13.56
3	V20C	1.90	6.43	5.14
4	V20D	9.20	74.02	28.29
5	V20E	8.30	39.95	32.57
6	V20F	0.60	0.00	6.29
7	V20G	7.00	1.00	1.16
8	V20H	27.70	14.81	24.03
9	V20J	0.00	0.00	0.42
10	V70A	0.00	0.00	0.10
11	V70B	0.00	0.00	0.00
12	V70C	1.40	2.66	14.97
13	V70E	0.90	6.97	0.80
14	V70D	1.00	1.14	1.15
15	V70F	4.90	8.19	6.31
16	V70G	21.20	15.19	4.23
17	V13A	0.00	0.00	0.00
18	V13C	16.50	57.92	34.53
19	V13B	17.90	61.50	26.36
20	V13D	5.00	65.92	32.82
21	V13E	25.40	50.83	28.77
22	V11G	0.00	0.00	0.00
23	V11H	0.00	5.66	2.60
24	V11B	0.00	0.00	0.15
25	V11E	0.00	0.00	0.00
26	V11A	0.00	0.00	0.71
27	V11C	0.00	0.00	4.08
28	V11D	0.00	6.46	1.84
29	V11F	0.00	37.77	14.43
30	V11J	0.00	41.21	26.22
31	V11K	0.00	0.00	0.82
32	V11L	21.20	53.90	17.30
33	V11M	0.50	5.46	11.35
34	V12D	0.00	0.00	0.13
35	V12E	1.60	1.53	0.39
36	V12A	0.80	6.38	1.31
37	V12B	0.00	0.84	6.28
38	V12C	0.40	0.00	0.02
39	V12F	0.30	2.80	4.66

QC No.	Quaternary Catchment	Irrigated Area (km ²)		
		Midgley	CSIR (1999)	WARMS
40	V12G	0.00	2.48	1.53
41	V14A	26.10	22.14	14.38
42	V14B	1.30	3.13	4.38
43	V14E	1.30	1.56	0.00
44	V14C	0.00	10.02	3.77
45	V14D	2.40	9.05	4.04
46	V60A	3.40	0.61	1.00
47	V60B	15.00	3.47	4.91
48	V60C	2.60	0.00	0.00
49	V60D	0.00	0.00	1.68
50	V60E	0.00	2.08	4.78
51	V60F	1.30	0.00	0.00
52	V60G	0.00	1.65	1.96
53	V60H	0.00	0.00	0.79
54	V60J	0.00	0.00	0.00
55	V60K	0.00	0.00	0.05
56	V31E	9.20	9.18	27.34
57	V31G	1.60	0.53	1.70
58	V31F	0.00	1.08	7.28
59	V31H	0.00	1.23	1.50
60	V31J	0.00	1.44	2.55
61	V31K	2.50	3.14	2.86
62	V31A	5.00	0.58	1.08
63	V31B	5.90	0.00	3.38
64	V31C	2.40	1.60	11.54
65	V31D	7.50	6.89	18.09
66	V32A	0.00	0.00	0.00
67	V32B	4.30	6.73	10.97
68	V32C	1.70	0.00	1.78
69	V32D	12.10	4.24	27.39
70	V32E	1.60	0.00	17.44
71	V32F	6.00	3.93	1.32
72	V32G	0.00	1.23	21.00
73	V32H	0.00	0.21	23.89
74	V33A	0.00	2.78	2.71
75	V33B	0.00	0.00	0.00
76	V33C	0.00	0.00	2.08
77	V33D	0.00	0.00	0.00
78	V40A	0.00	0.00	0.00
79	V40B	0.00	0.00	0.00
80	V40C	0.00	0.00	0.00
81	V40D	0.00	0.00	0.00
82	V40E	0.00	0.00	0.05
83	V50A	0.00	0.00	0.00
84	V50B	0.00	0.00	1.63
85	V50C	0.00	0.00	1.95
86	V50D	0.00	0.00	4.55
Total		294.90	701.49	593.80

4.3. Assessment of Observed Flow Data

The Thukela Catchment is an operational catchment with flow data monitored and distributed by DWAF. Before verification of simulated results, it was necessary to assess the reliability of the observed flow data, identify errors and anomalies in the data, and also to identify periods of data which can be used for verification. Past experience and an examination of primary flow data obtained from DWAF indicated that, at some stations, the discharge : stage rating curve is frequently exceeded. Thus, the rating curves were extended using a methodology similar to that outlined by van Rensburg (2005). Checks for periods of missing data and the relationship between observed runoff and rainfall were used to assess the observed flow records at annual levels.

4.3.1. Extension of Rating Tables

It was assumed that the threshold in discharge noted in the primary data from some gauging weirs, an example of which is shown in Figure 4.6, is the result of the rating table for weir being exceeded. While the entire gauging structure could have been overtopped during these threshold events, this is unlikely as data were recorded during the events. This limitation of the flow data will result not only in incorrect flow volumes, but will also influence the Annual Maximum Series (AMS) which are used in this study to estimate design floods.

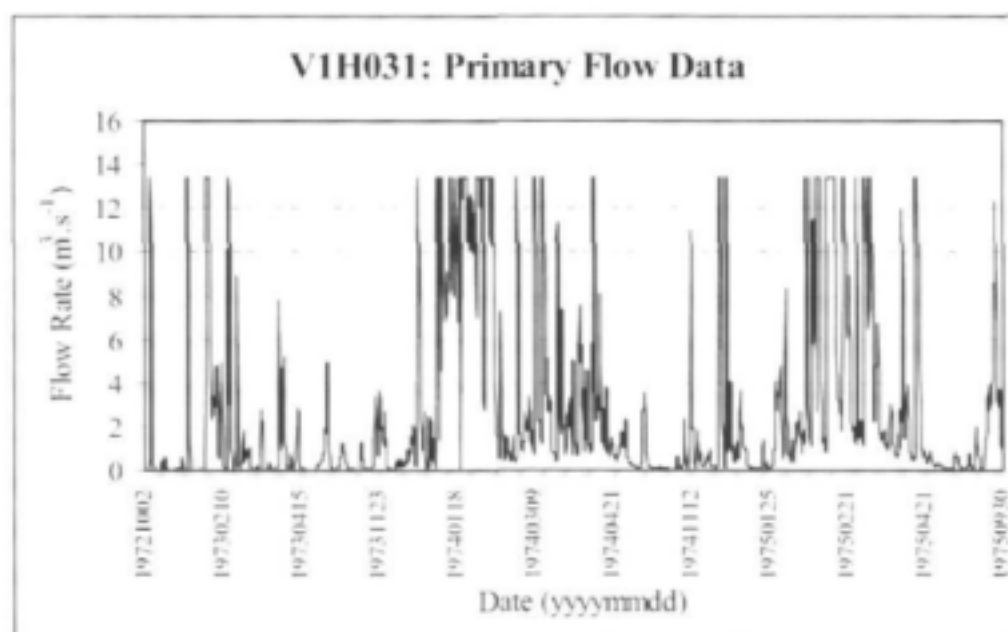


Figure 4.6 Example of a maximum flow threshold in the primary flow data as a result of a limited rating table

At gauging stations where evidence of limited rating tables were noted, a simple log-log extension of the rating table was performed, as described by van Rensburg (2005). The extended rating table was then used to recalculate the flow rates from the recorded stage heights. The flow gauging stations in the Thukela Catchment where the rating tables were extended in this project are listed in Table 4.2.

Table 4.2 Flow gauging stations in the Thukela Catchment where the rating tables were extended

Station Name	
V1H001	V3H007
V1H026	V3H009
V1H031	V3H010
V1H041	V3H011
V1H058	V3H027
V2H005	V5H002
V2H006	V7H016
V3H002	

An example of a rating table is plotted in Figure 4.7. Extension of the rating table was then achieved by extending the log plot of the rating table using a straight line fitted to the last straight line segment of the rating table. The extension is achieved by conducting a regression analysis in log-space on the "last straight line segment" of the rating table, as depicted in Figure 4.8.

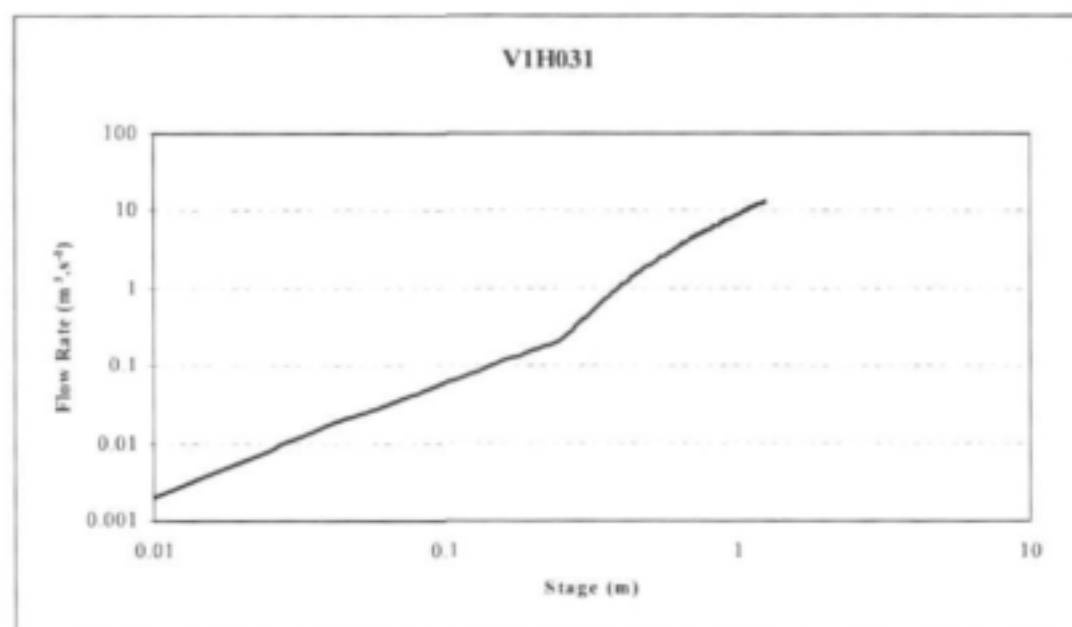


Figure 4.7 Log-log representation of rating table at V1H031

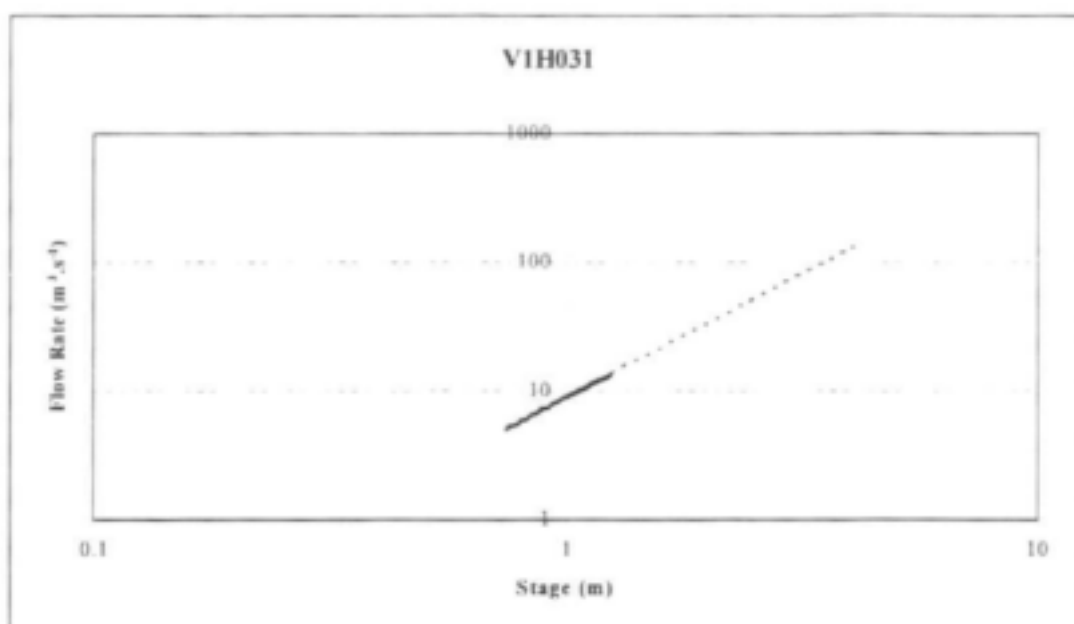


Figure 4.8 Extension of last straight-line segment of log-log representation of rating table at V1H031

Extension of rating tables was done using a straight line approach described above. This approach is based on the assumption of relatively uniform river sections above the rating table limit, i.e. the increase in the flow area above the rating table limit will follow the same trend as was the case over the last straight line portion of the rating table (van Rensburg, 2005). Underestimation of the actual flow rate can take place if there is a sudden increase in flow area. In this case, the increased flow area would result in a higher increase in the flow rate per incremental height increase, than below the threshold of the flood plain, hence resulting in a different slope of the rating table line (van Rensburg, 2005).

Following from the above, it is clear that log-extension of rating tables can only be done within certain bounds, as extensive extrapolation above the known rating table limit will result in increased uncertainty regarding the estimated flows.

The accumulated streamflow for two scenarios are shown in Figure 4.9. The blue line represents the accumulated streamflow over a 30 year period where the rating table does not cover the entire range of recorded flow stages and the red line represents the accumulated streamflow at the same station where the rating table has been extended using the abovementioned method. Figure 4.10 illustrates the effect on the AMS of extending the rating table at a gauging weir which has a limited rating table. It is thus evident that a limited rating table will result in the underestimation of both the volume and flood peak discharges.

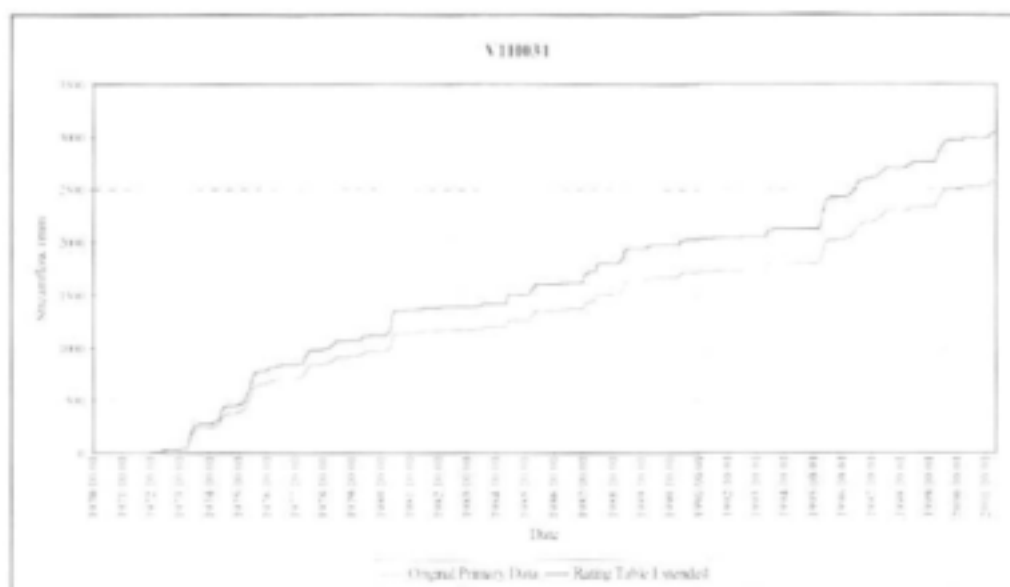


Figure 4.9 Accumulated flow over a 30 year period at Weir V1H031 using the original primary data and primary data derived from the extended rating table

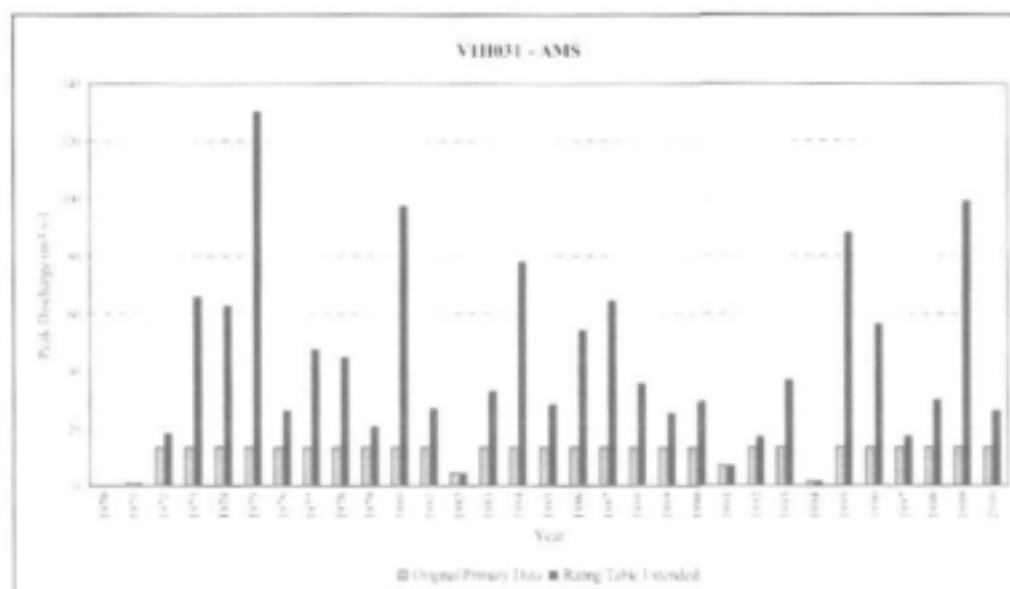


Figure 4.10 AMS for Weir V1H031 extracted from the original primary data and from primary data derived from the extended rating table

4.3.2. Reliability of Observed Flow Data

In order to obtain a general assessment of the reliability of the observed flow data, annual runoff : rainfall ratios and percentages of missing flow data were compared, as shown in Figures 4.11 to 4.39, using the selected raingauge for the sub-catchment where the gauging weir is located. In case where the observed data appeared to be suspect, a second raingauge located in either in, or in close proximity to, the sub-catchment was also included in the analysis. A summary of the assessment of the reliability of the flow data is contained in Table 4.3.

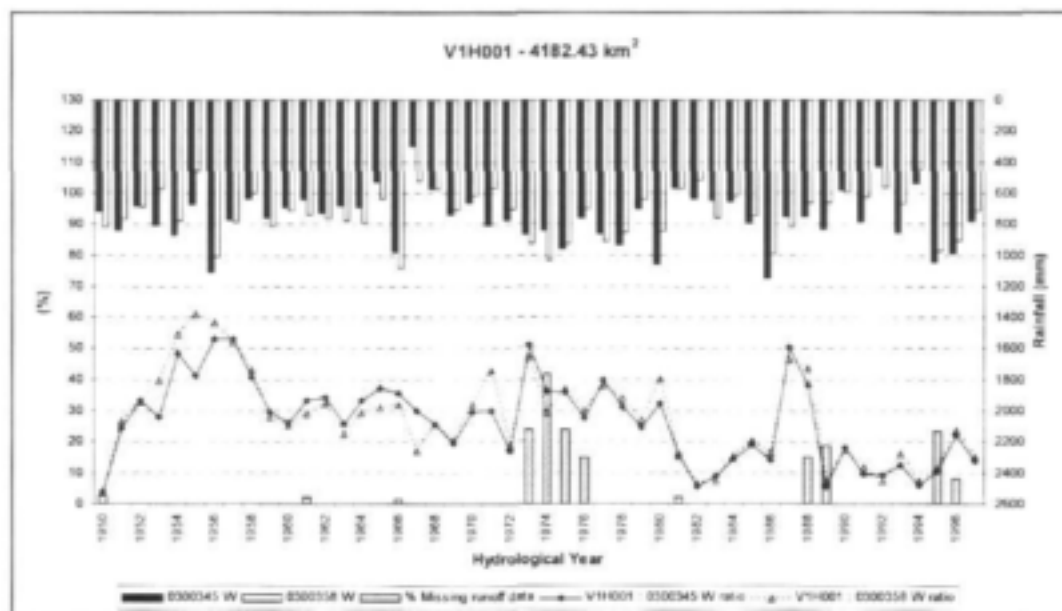


Figure 4.11 Analysis of annual flow data at Weir V1H001

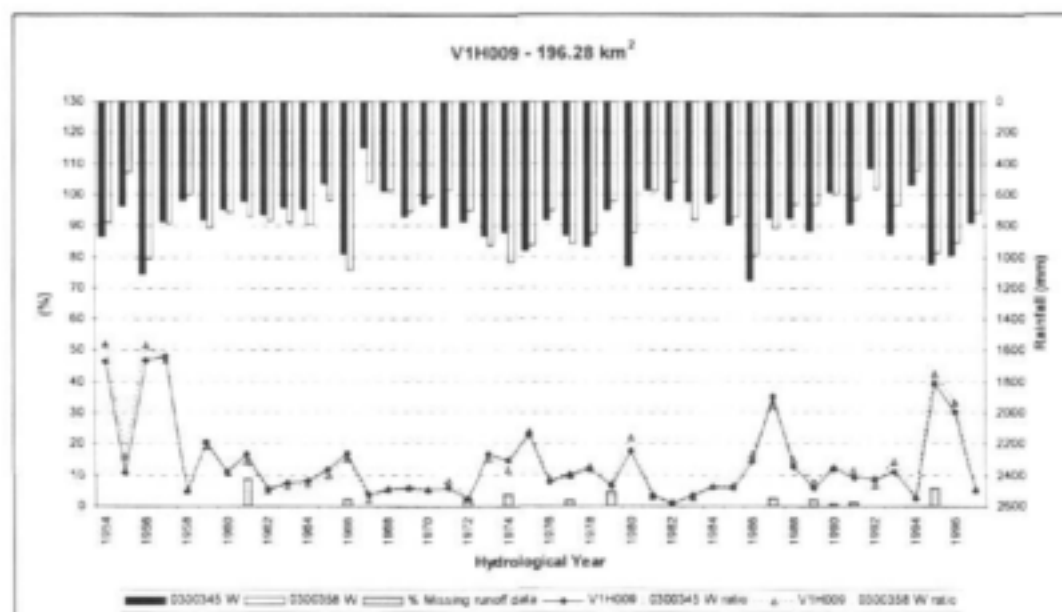


Figure 4.12 Analysis of annual flow data at Weir V1H009

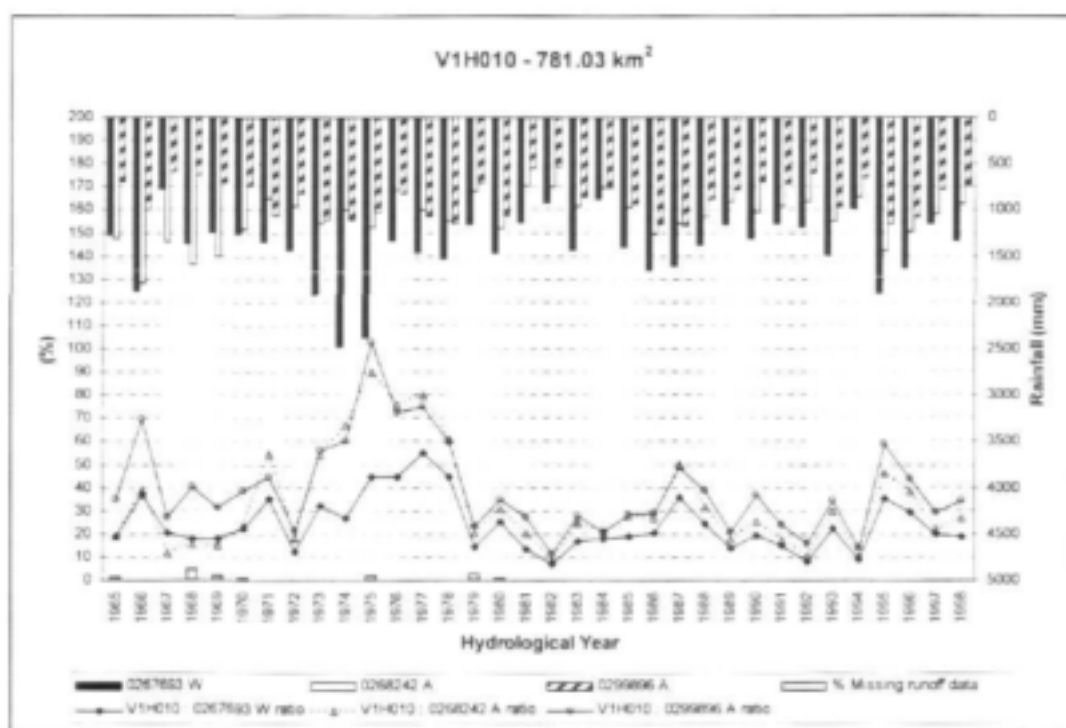


Figure 4.13 Analysis of annual flow data at Weir V1H010

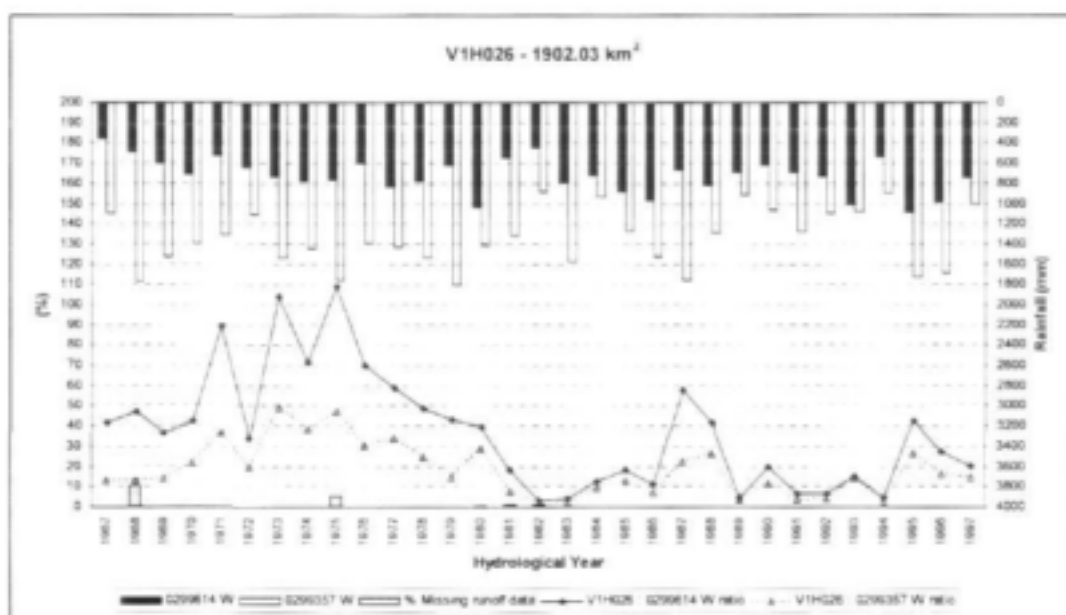


Figure 4.14 Analysis of annual flow data at Weir V1H026

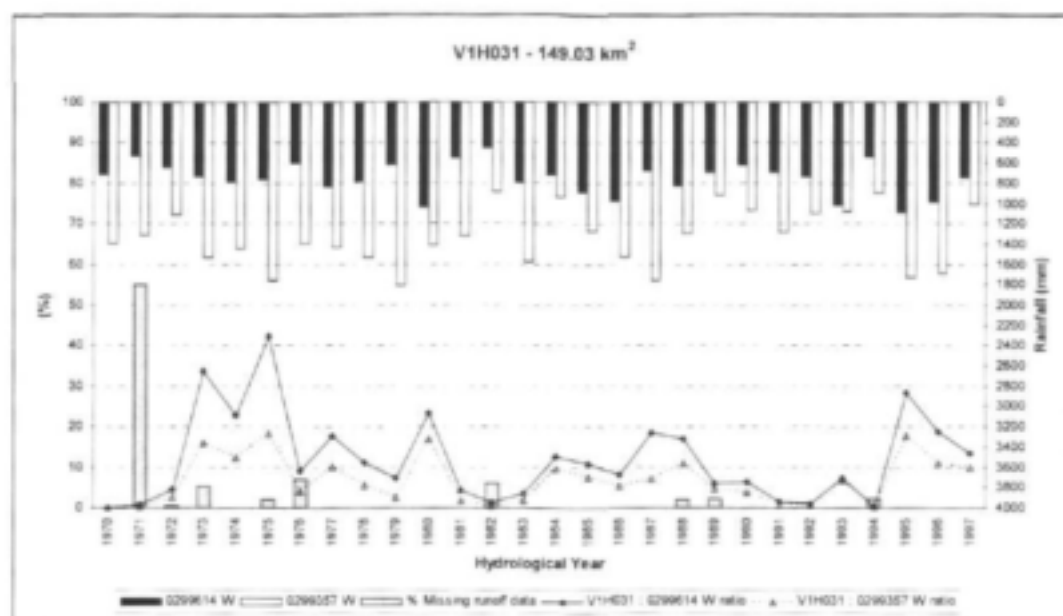


Figure 4.15 Analysis of annual flow data at Weir V1H031

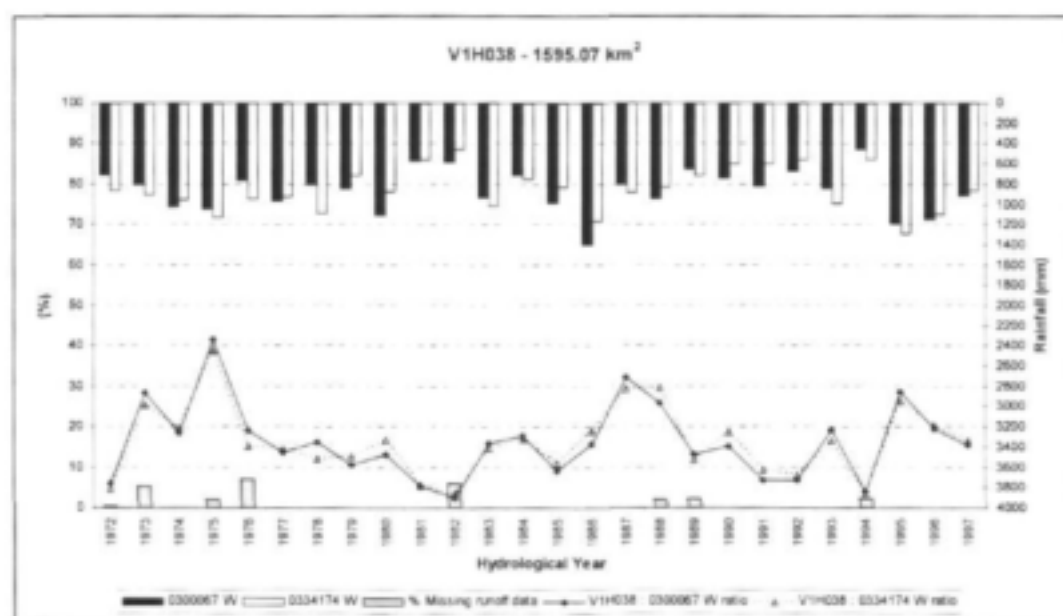


Figure 4.16 Analysis of annual flow data at Weir V1H038

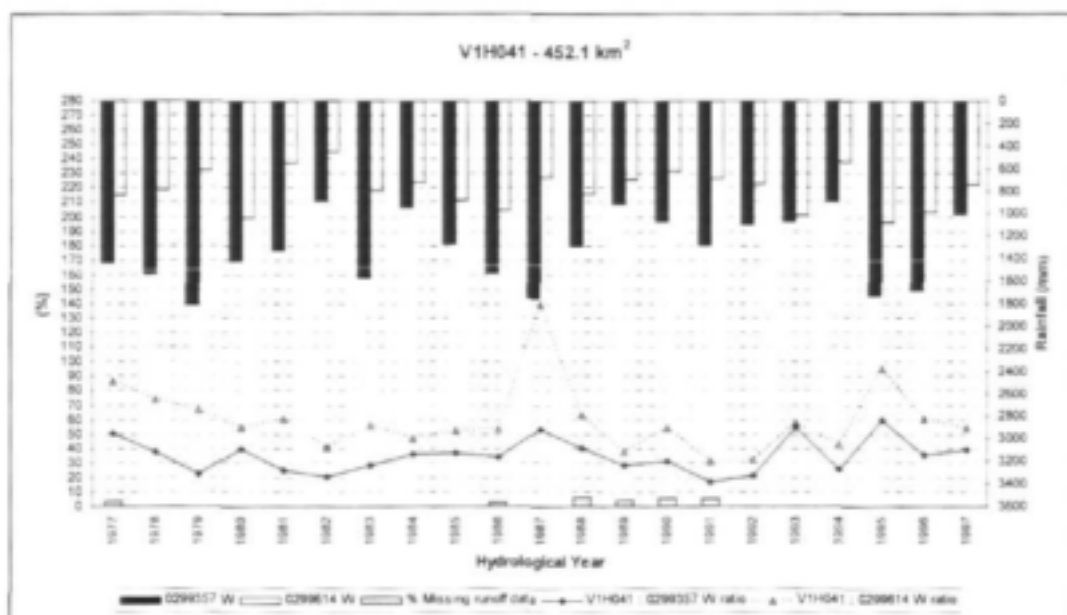


Figure 4.17 Analysis of annual flow data at Weir V1H041

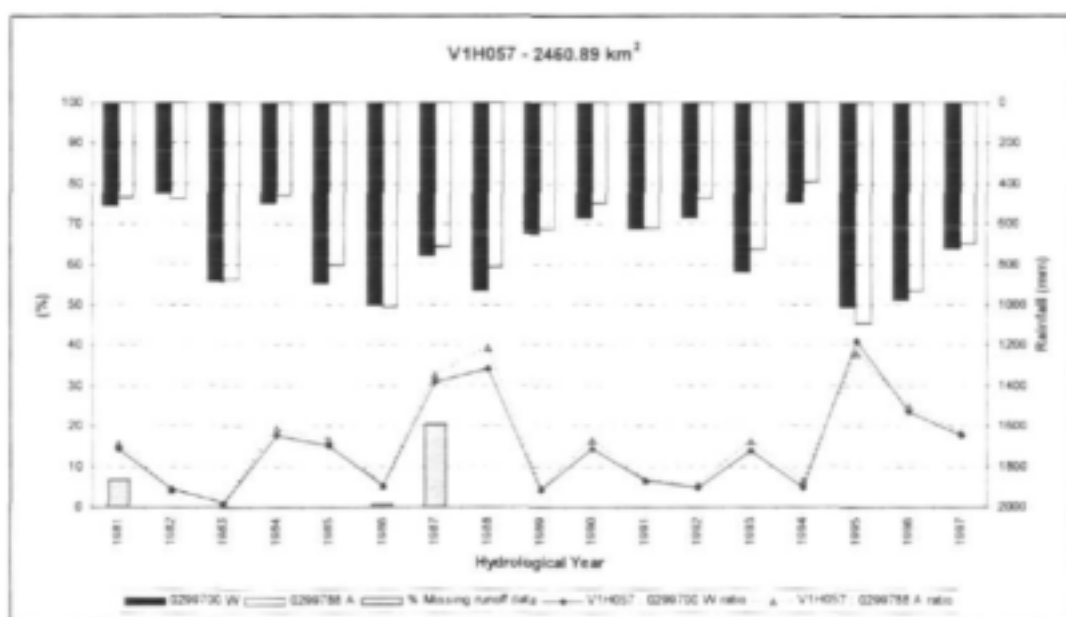


Figure 4.18 Analysis of annual flow data at Weir V1H057

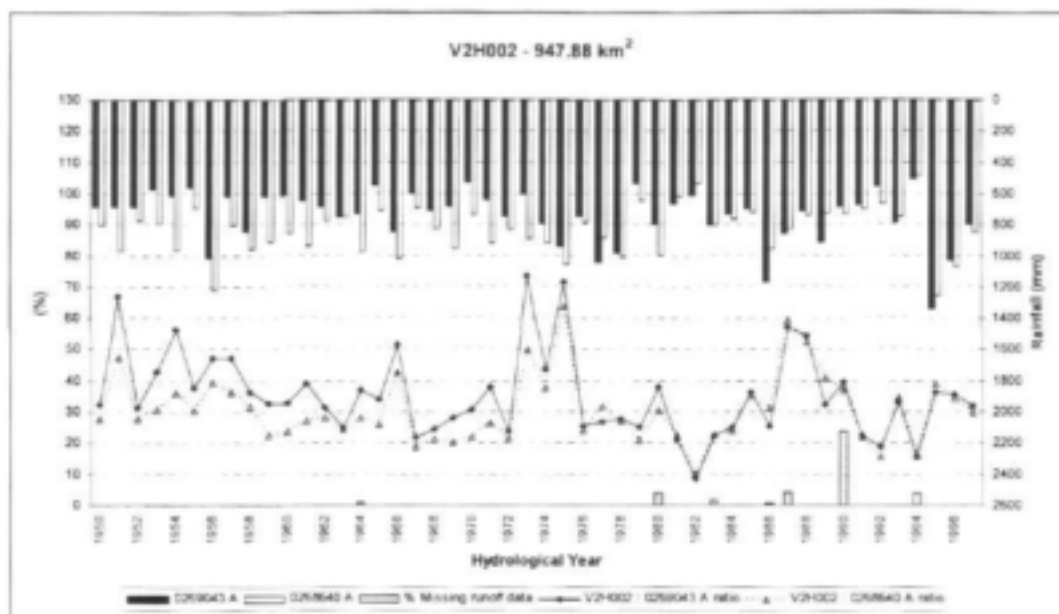


Figure 4.19 Analysis of annual flow data at Weir V2H002

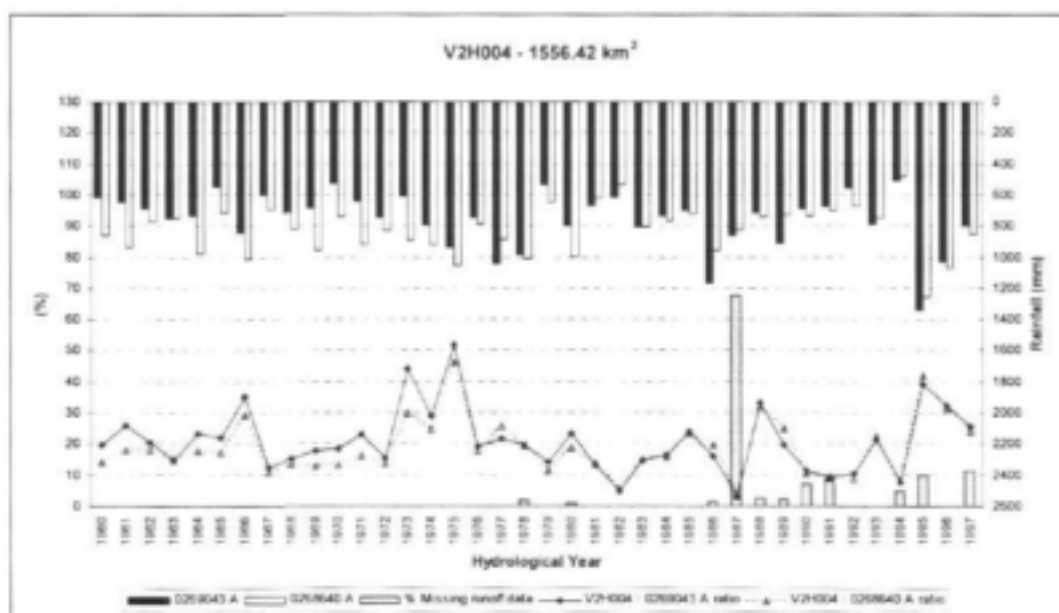


Figure 4.20 Analysis of annual flow data at Weir V2H004

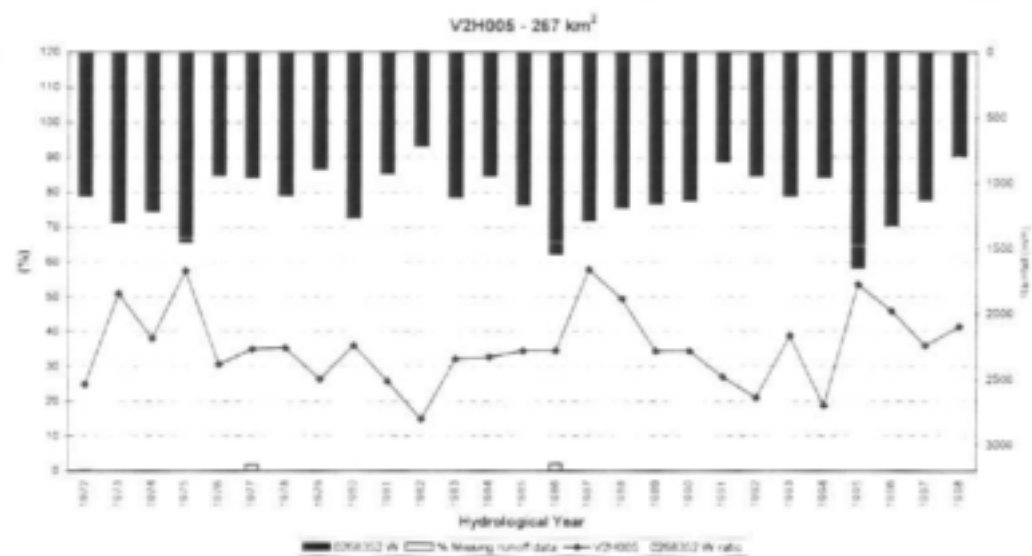


Figure 4.21 Analysis of annual flow data at Weir V2H005

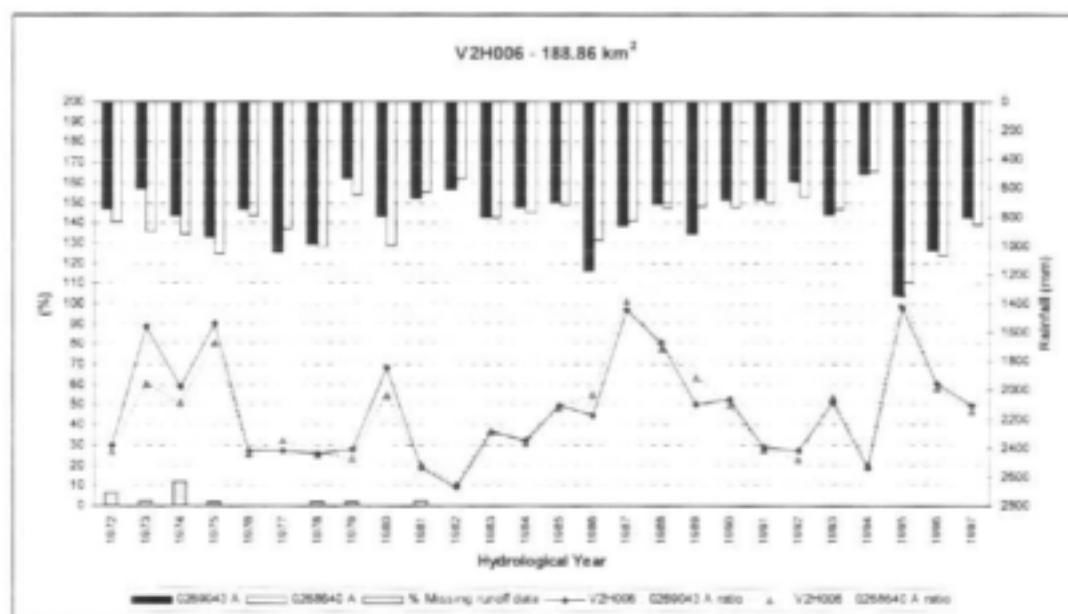


Figure 4.22 Analysis of annual flow data at Weir V2H006

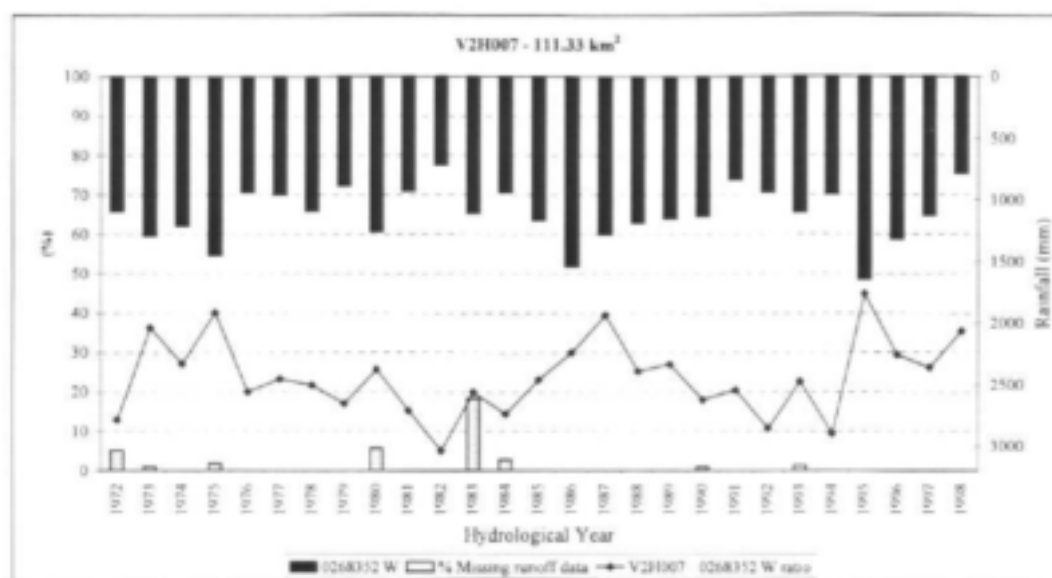


Figure 4.23 Analysis of annual flow data at Weir V2H007

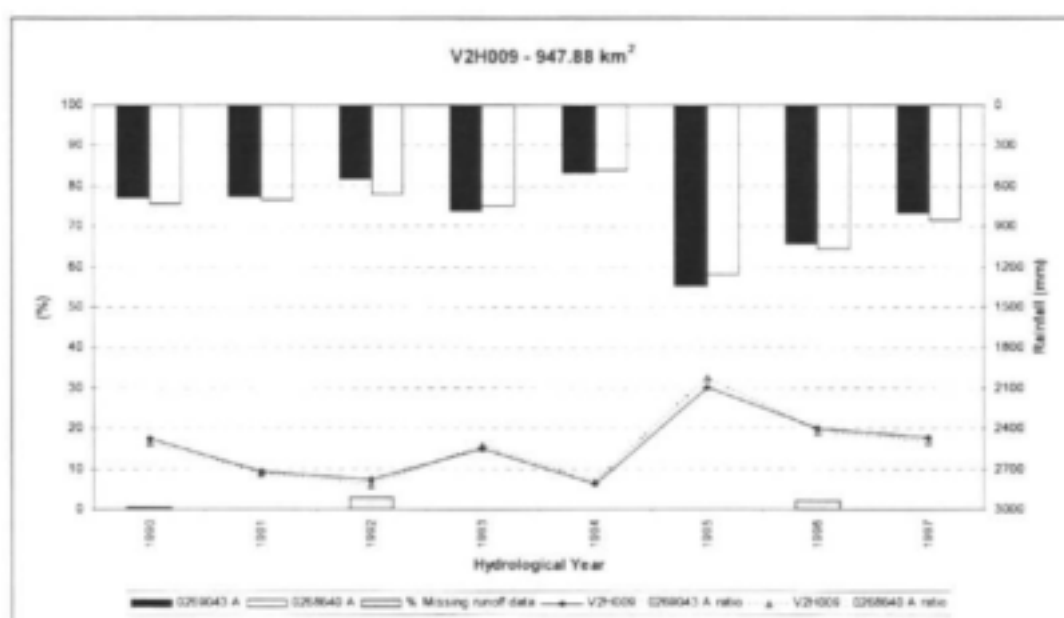


Figure 4.24 Analysis of annual flow data at Weir V2H009

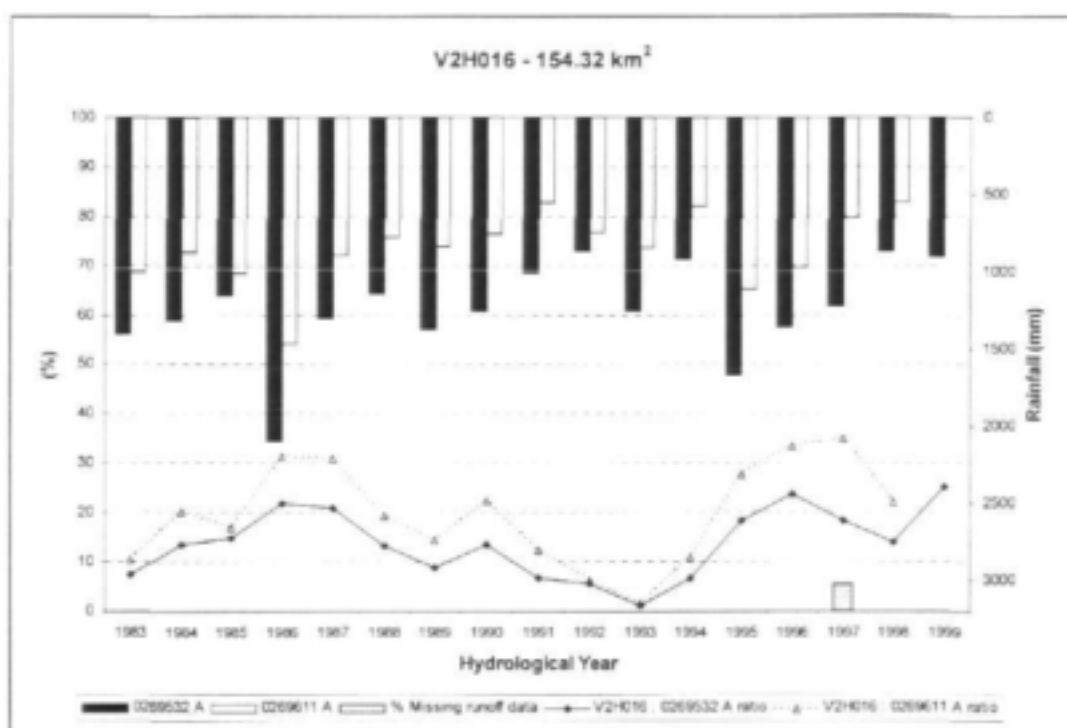


Figure 4.25 Analysis of annual flow data at Weir V2H016

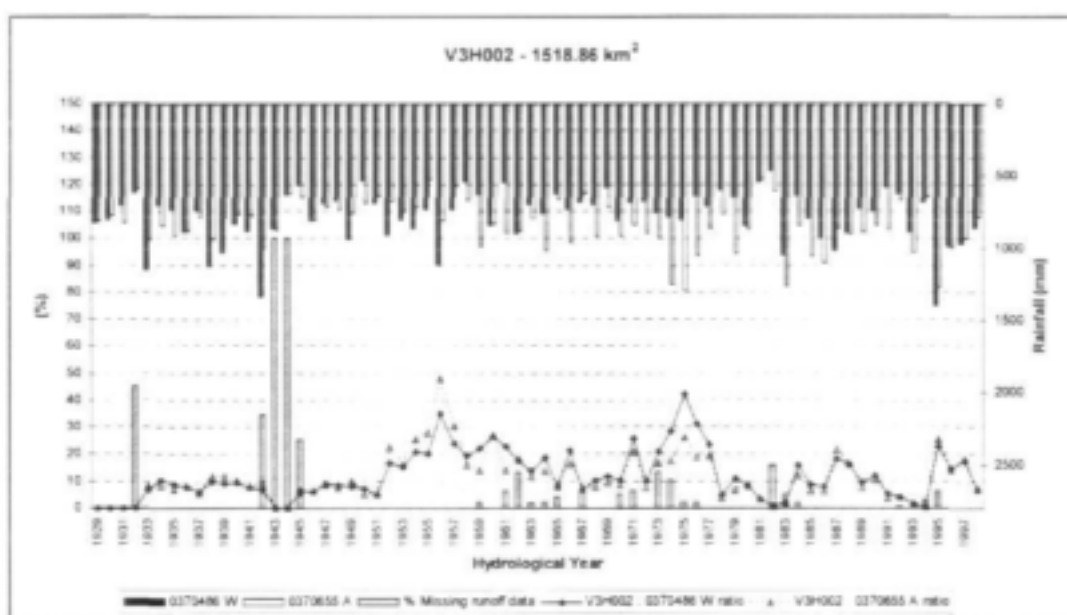


Figure 4.26 Analysis of annual flow data at Weir V3H002

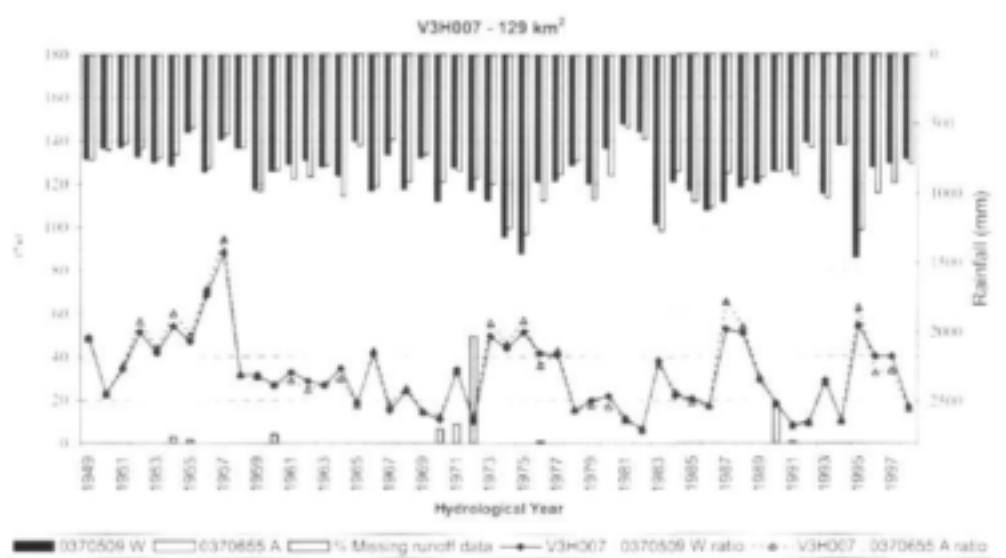


Figure 4.27 Analysis of annual flow data at Weir V3H007

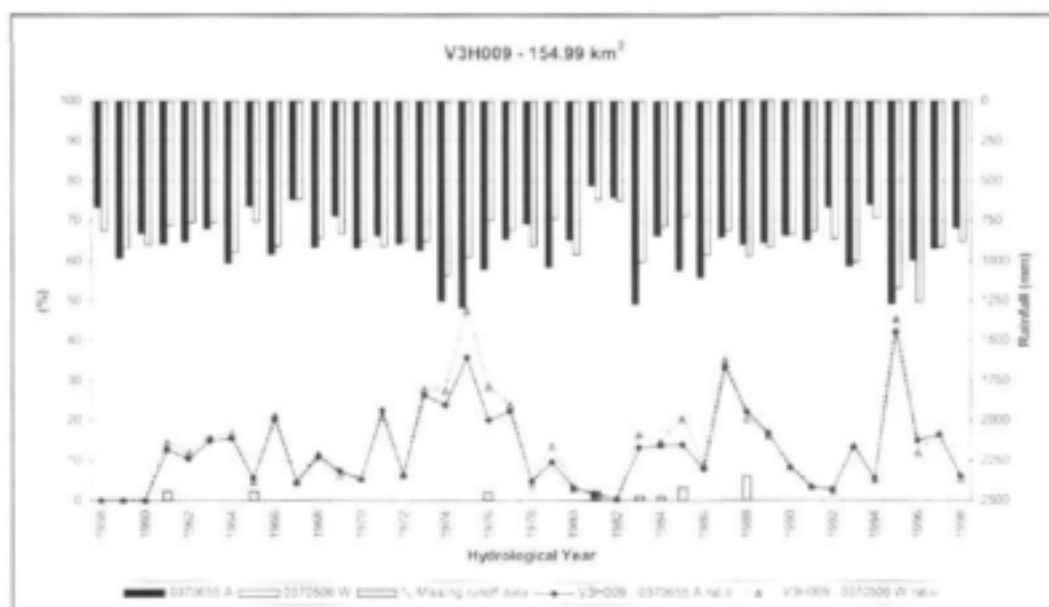


Figure 4.28 Analysis of annual flow data at Weir V3H009

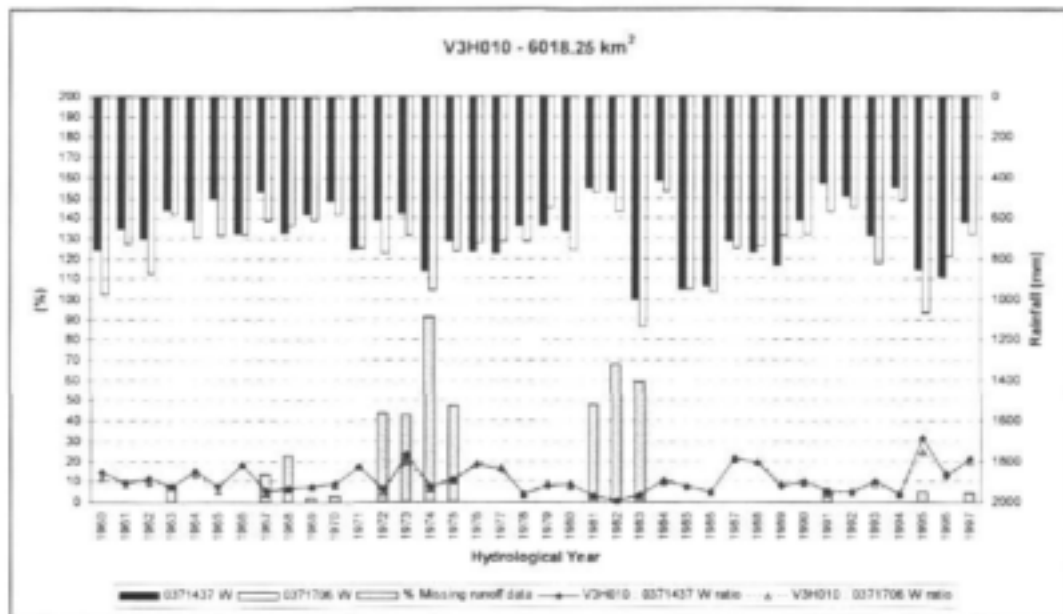


Figure 4.29 Analysis of annual flow data at Weir V3H010

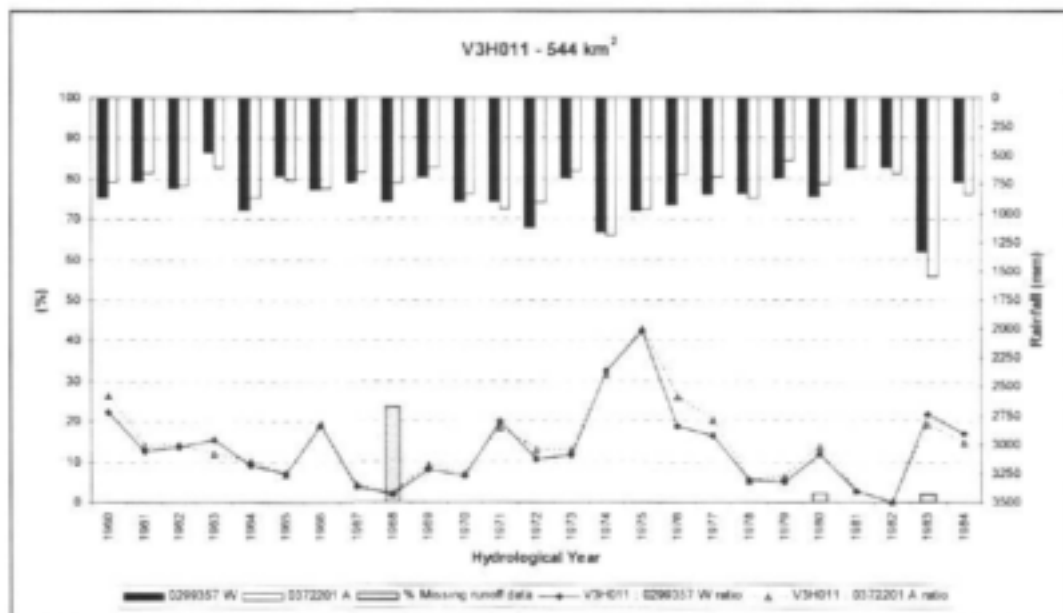


Figure 4.30 Analysis of annual flow data at Weir V3H011

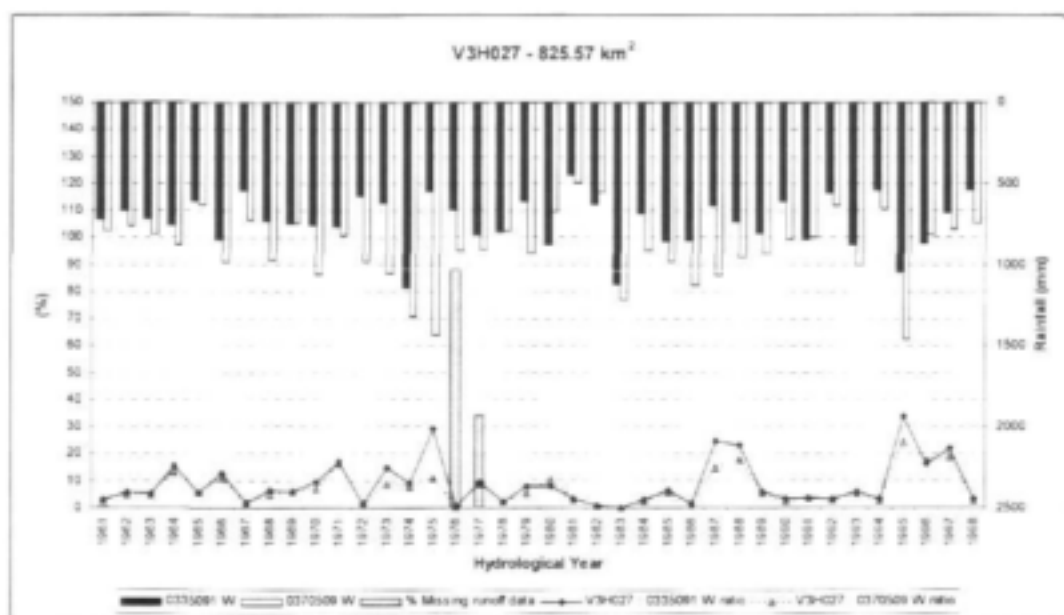


Figure 4.31 Analysis of annual flow data at Weir V3H027

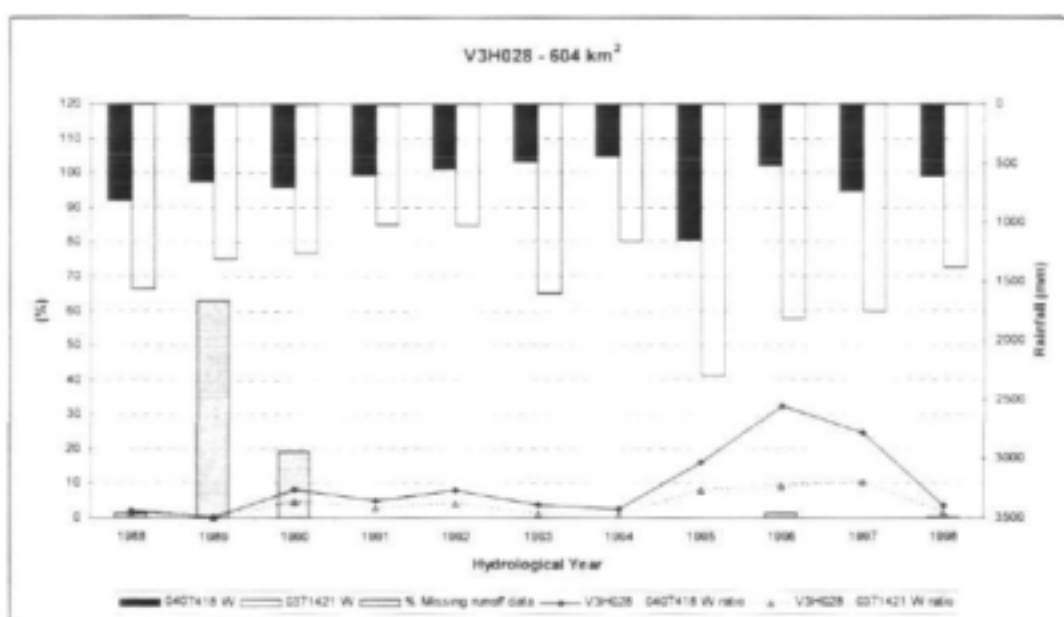


Figure 4.32 Analysis of annual flow data at Weir V3H028

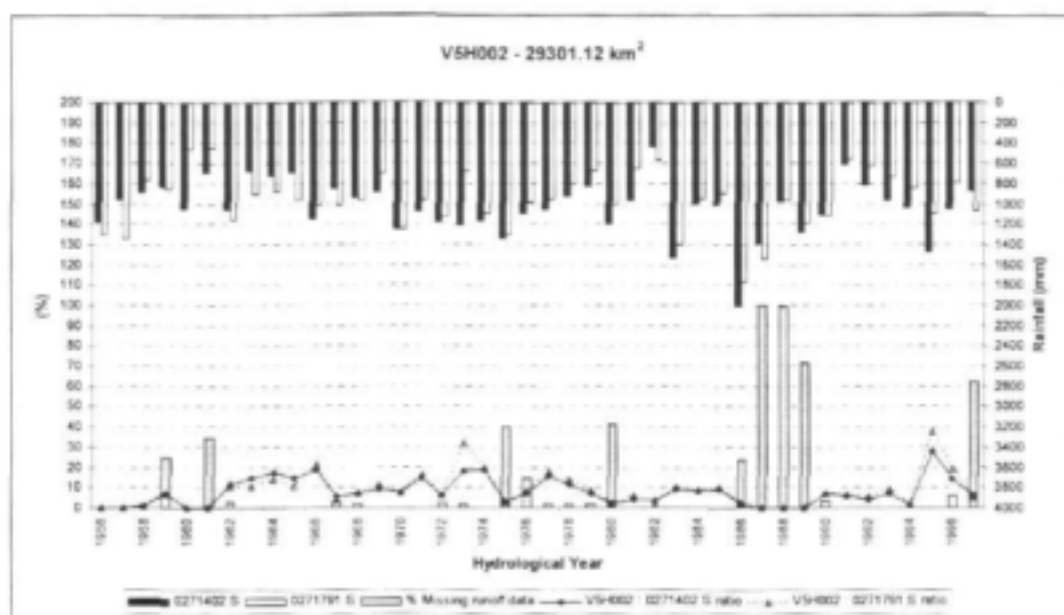


Figure 4.33 Analysis of annual flow data at Weir V5H002

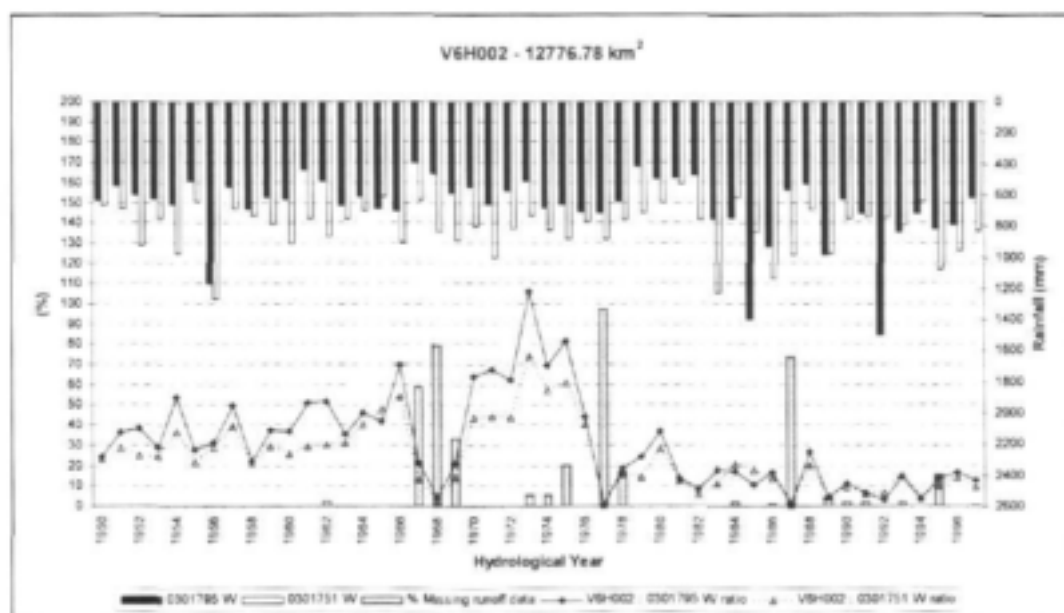


Figure 4.34 Analysis of annual flow data at Weir V6H002

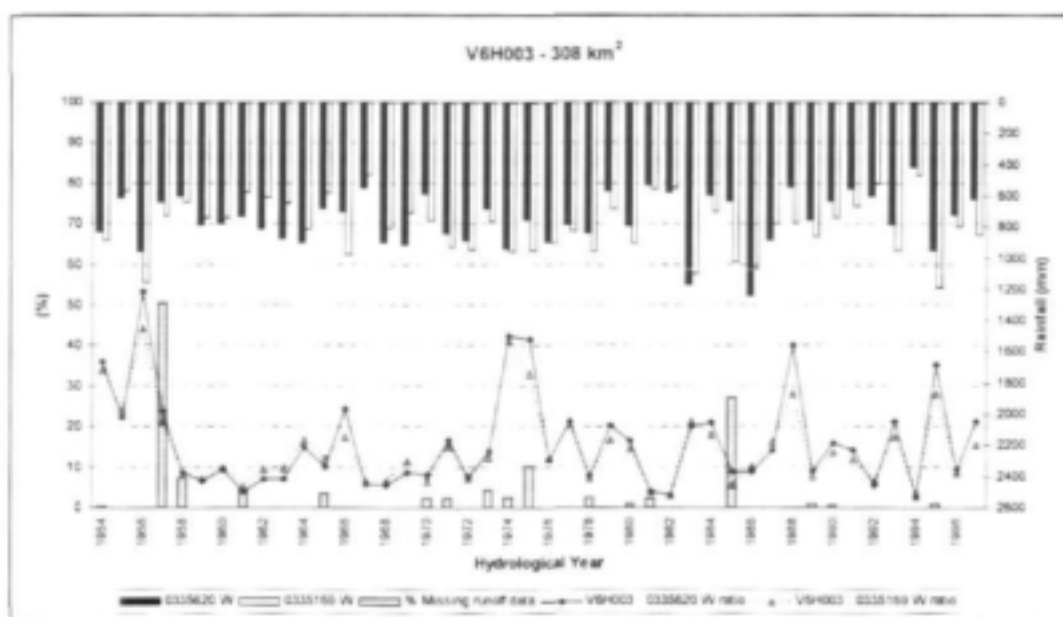


Figure 4.35 Analysis of annual flow data at Weir V6H003

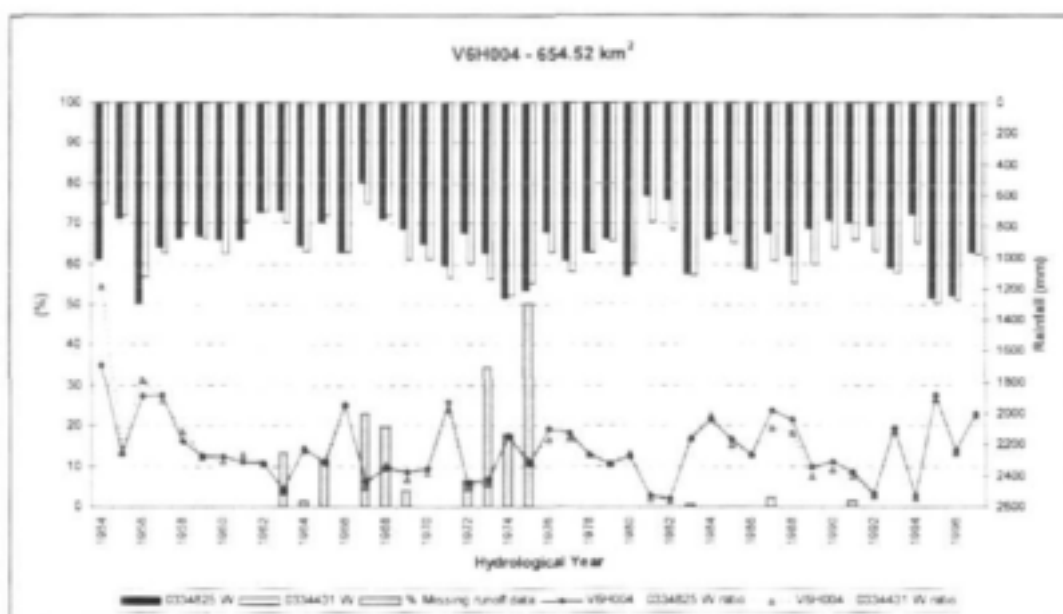


Figure 4.36 Analysis of annual flow data at Weir V6H004

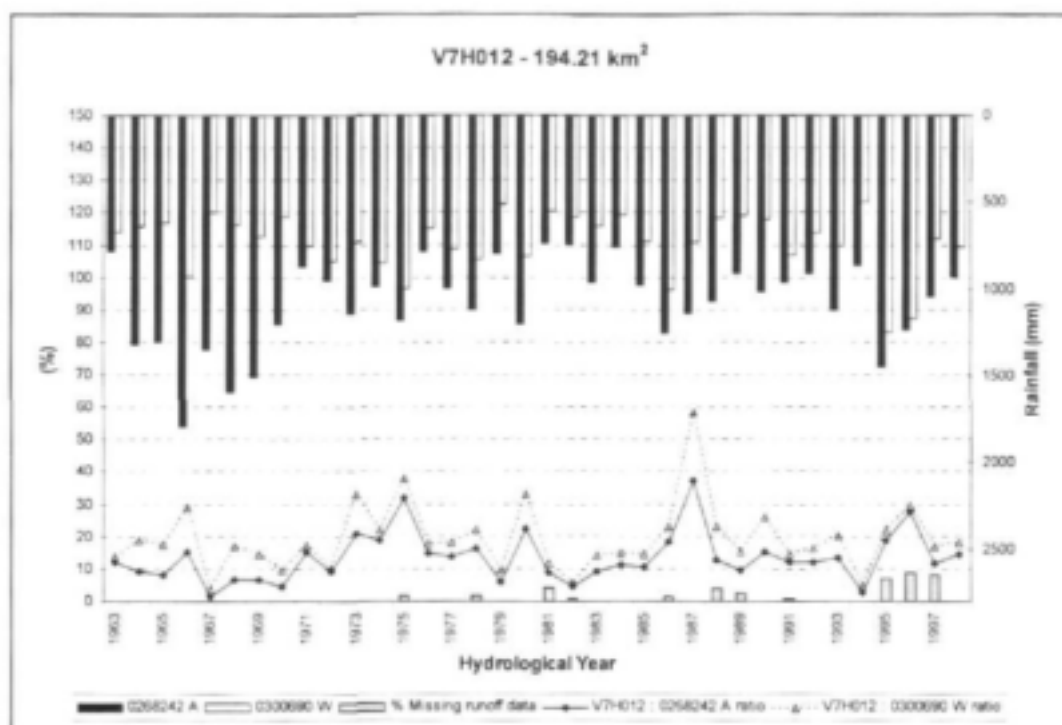


Figure 4.37 Analysis of annual flow data at Weir V7H012

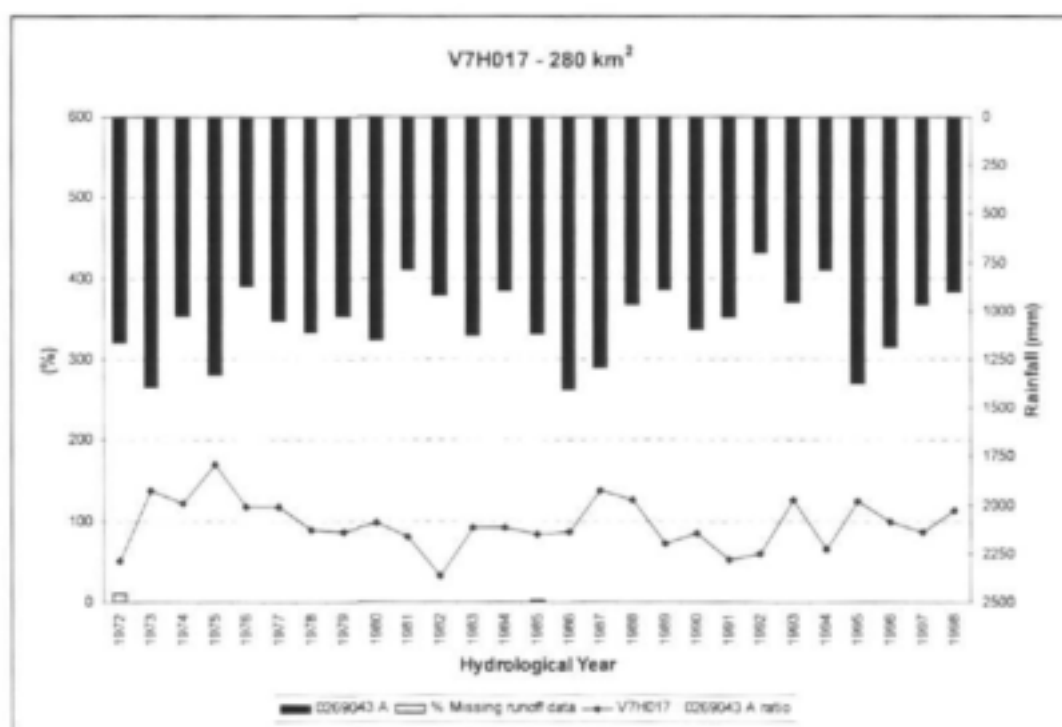


Figure 4.38 Analysis of annual flow data at Weir V7H017

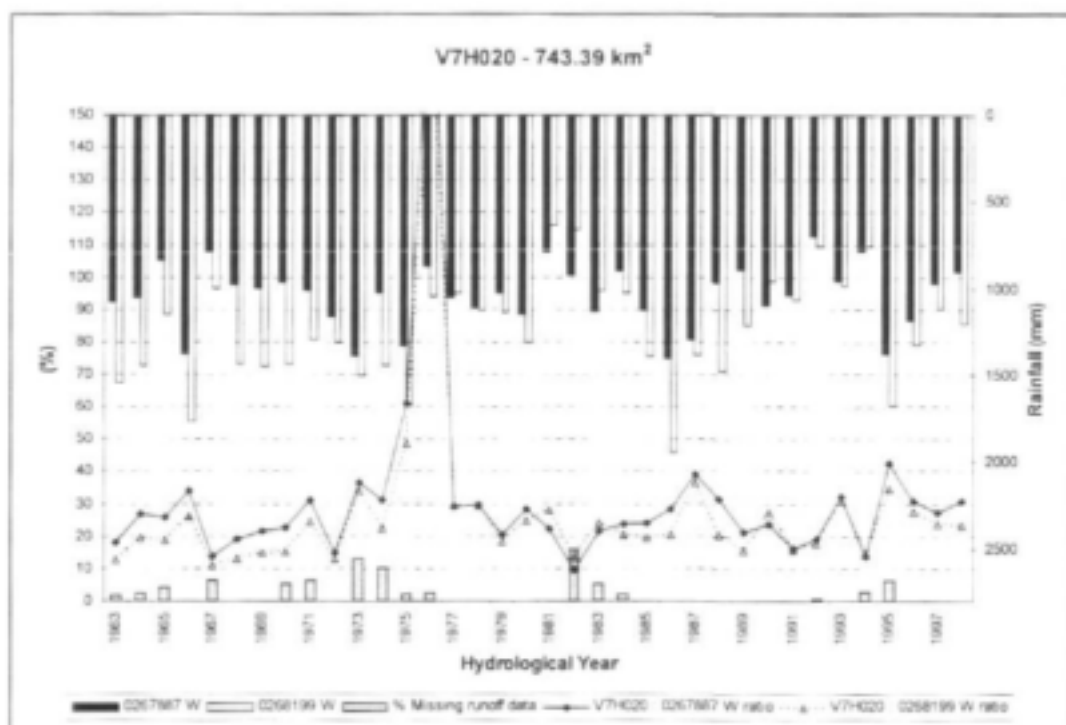


Figure 4.39 Analysis of annual flow data at Weir V7H020

Table 4.3 Summary of weir analyses and assessment of data

Weir	Comments	Use of Data
V1H001	Little runoff relative to rainfall in 1950, construction of upstream dam? More than 10% data missing in 1973 - 1976, 1988, 1989 and 1995.	Data not acceptable for verification
V1H009	No overtopping, very few missing data	Use entire record
V1H010	Runoff:rainfall ratios from 1972 to 1979 appear too high. Generally few missing data.	Exclude data prior to 1980
V1H026	Rating table extended. Few missing data. Runoff:rainfall ratio high 1971-1978 for 0299614W raingauge and variable rainfall in catchment.	Use entire record
V1H031	Little missing data after 1971	Use from 1972 onwards
V1H038	No overtopping. Few missing data.	Use entire record
V1H041	No overtopping. Few missing data.	Weir not located at catchment outlet
V1H057	No overtopping. Flow data suspect prior to 1987. More than 10% missing data in 1987.	Use from 1988 onwards
V2H002	No overtopping. More than 20% missing in 1990.	Best period 1950 - 1989
V2H004	No overtopping. Missing data from 1987 onwards.	Use 1950 - 1986
V2H005	No overtopping. Few missing data.	Use entire record

Weir	Comments	Use of Data
V2H006	Rating table frequently exceeded. Very high runoff:rainfall ratios from both raingauges possibly caused by extension of rating table or inter-catchment transfer of water	Data not acceptable for verification
V2H007	No overtopping. Few missing data/	Use entire record
V2H009	No overtopping	Use entire record
V2H016	Delayed response noted, particularly 1991-1995. Due to dams or irrigation?	Use entire record
V3H002	Data poor prior to 1959 (stick gauge). Significant missing data.	Data not acceptable for verification
V3H007	Missing data generally in winter months. Rating table changed in 1955, overtopped prior to this.	Use from 1958 onwards, excluding 1972 where > 40% data missing
V3H009	Not much missing data. No flow prior to 1960.	Use 1962 to end of record
V3H010	Many missing data up to mid-1980s	Data not acceptable for verification
V3H011	Declining runoff:rainfall ratio due to dam construction or irrigation?	Use entire record, exclude 1968
V3H027	Rating table and data suspect before 1976 and > 30% missing data in 1976 and 1977.	Best period of record 1978-2000. Weir located downstream of Chelmsford Dam. Data not acceptable for verification.
V3H028	Much missing data prior to 1991. Low runoff:rainfall ratios accounted for by dam.	Best period 1991 – 1998. Weir downstream of Zaaikoek Dam. Data not acceptable for verification.
V5H002	Frequent overtopping and rating table extended. Many missing data.	Data not acceptable for verification.
V6H002	No overtopping. Much missing data. The change in runoff:rainfall ratio after 1977 could be as a result of new rating table.	Data not acceptable for verification.
V6H003	No overtopping. Some missing data. Flow in 1974 and 1975 not consistent with rainfall.	Exclude 1957 and 1985
V6H004	No overtopping. Many missing data prior to 1976.	Best period 1976-2000
V7H012	No runoff in 1967 despite > 1100 mm of rainfall	Best period 1968 - 1994
V7H017	Few missing data. Runoff:rainfall ratios unrealistic. May be due to inter-catchment transfer.	Data not suitable for verification.
V7H020	Few missing data. High releases 1973 – 1978.	Best period 1983-1993. Weir below Wagendrift dam. Data not acceptable for verification.

4.4. Estimation of Design Floods from Observed Data

Design floods were estimated by fitting probability distributions to the AMS of peak discharges. L-moments (Hosking, 1990; Hosking, 1991) were used to fit the candidate distributions to the AMS. These included, *inter alia*, the 2- and 3-parameter log-Normal (LN), log-Pearson-III (LP3) and General Extreme Value (GEV). Alexander (1990) recommends the LP3 distribution for South Africa while more recently Parak and Pegram (2005) used the GEV for design flood estimation in South Africa.

4.4.1. Comparison of Annual Maximum Series Derived from Observations

Two sources of AMS were utilised in the study. The first source extracted AMS from the primary flow data while the second source was a data file provided by van Bladeren (2000). As shown in Figure 4.40 to Figure 4.60, there is generally a close correspondence between the AMS extracted from the primary flow data, using extended rating tables where necessary, and the AMS supplied by van Bladeren (2000). Examples of where the use of the extended rating tables, as performed in this study, improve the flow data are shown in Figure 4.43, Figure 4.44, Figure 4.46, Figure 4.54 and Figure 4.59. Some of the differences in the AMS are a result of missing primary flow data. For example, primary data are missing during 1986 at Weir V2H004 (Figure 4.48) which explains the difference in the annual maximum streamflow for that year.

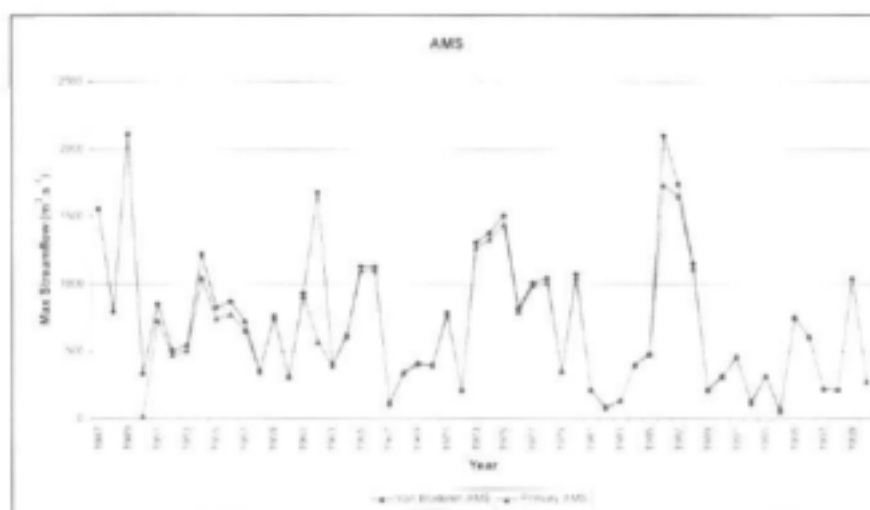


Figure 4.40 AMS at Gauging Weir V1H001

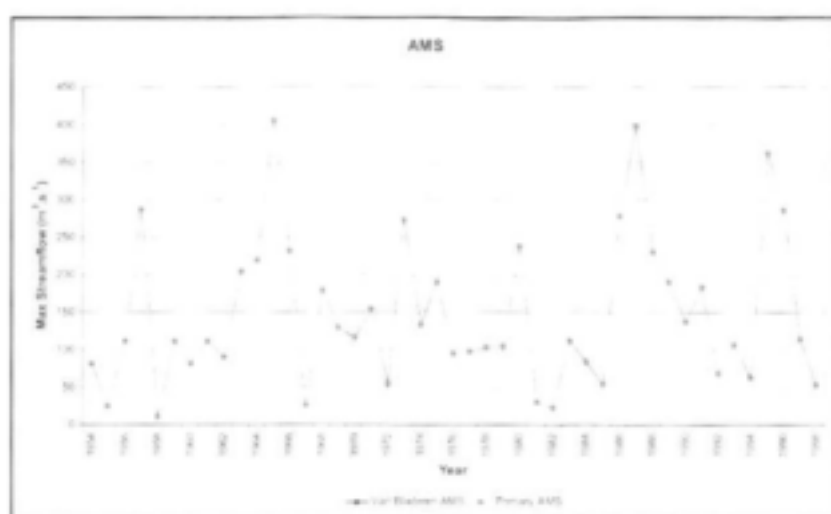


Figure 4.41 AMS at Gauging Weir VIH009

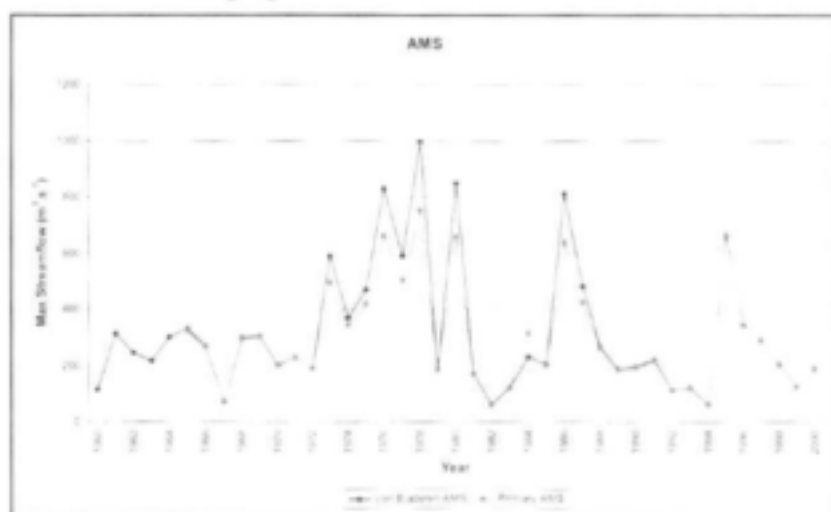


Figure 4.42 AMS at Gauging Weir VIH010

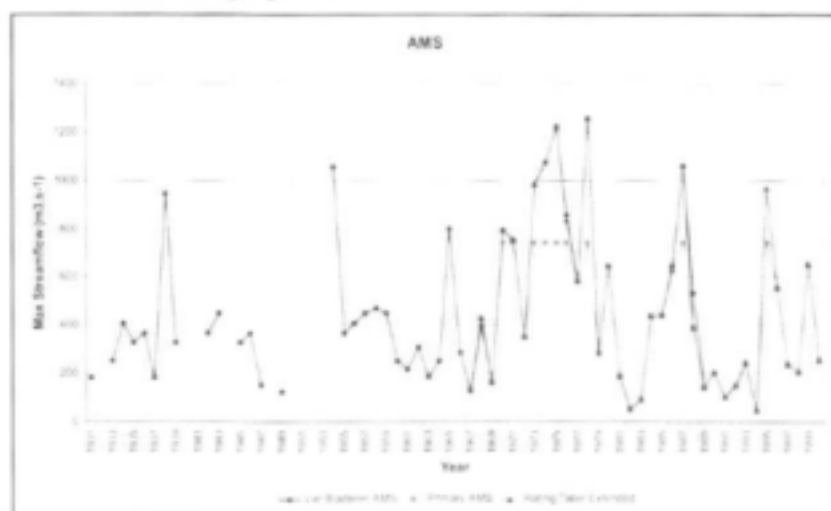


Figure 4.43 AMS at Gauging Weir VIH026

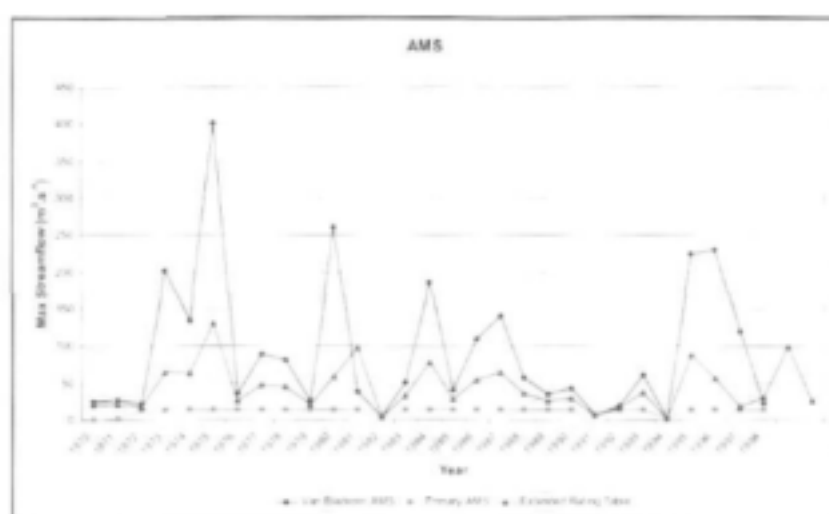


Figure 4.44 AMS at Gauging Weir VIH031

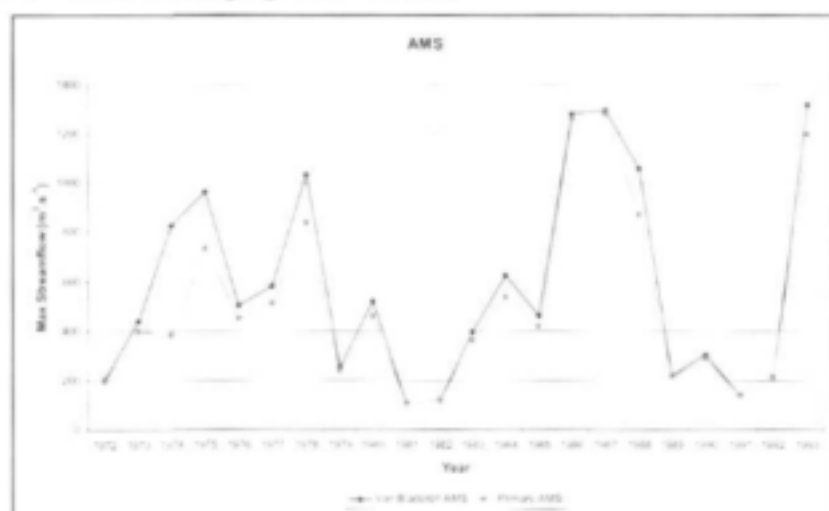


Figure 4.45 AMS at Gauging Weir VIH038

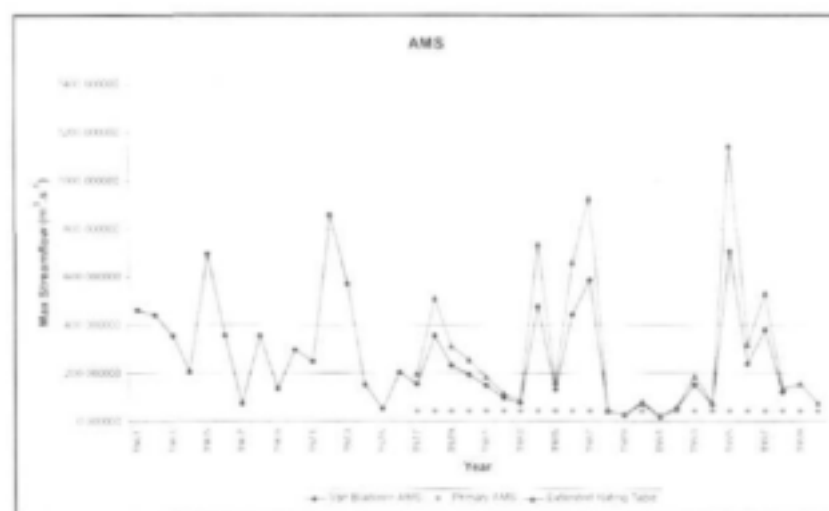


Figure 4.46 AMS at Gauging Weir VIH041

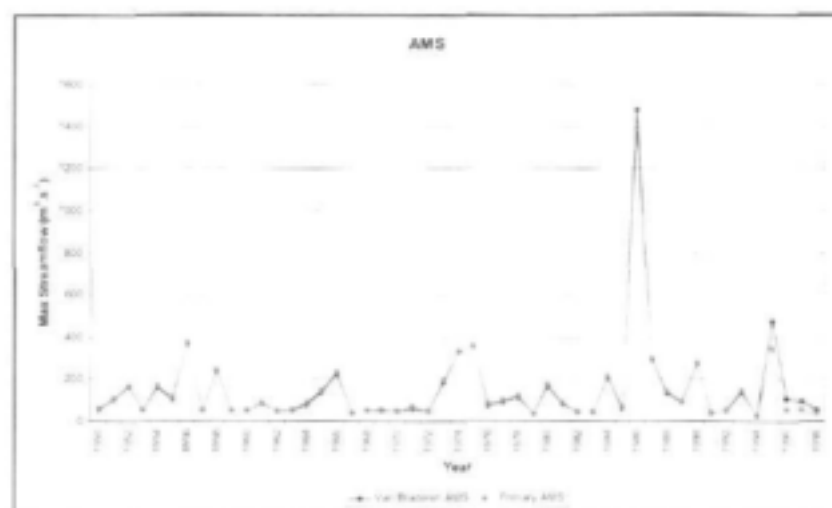


Figure 4.47 AMS at Gauging Weir V2H002

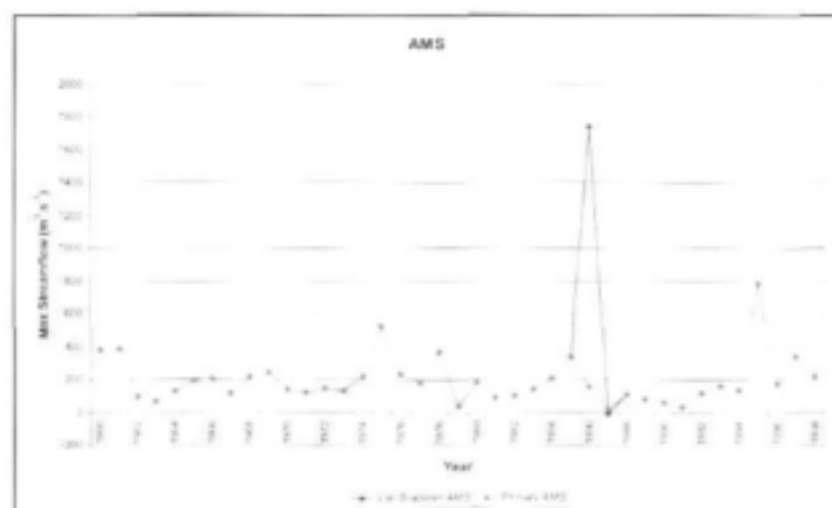


Figure 4.48 AMS at Gauging Weir V2H004

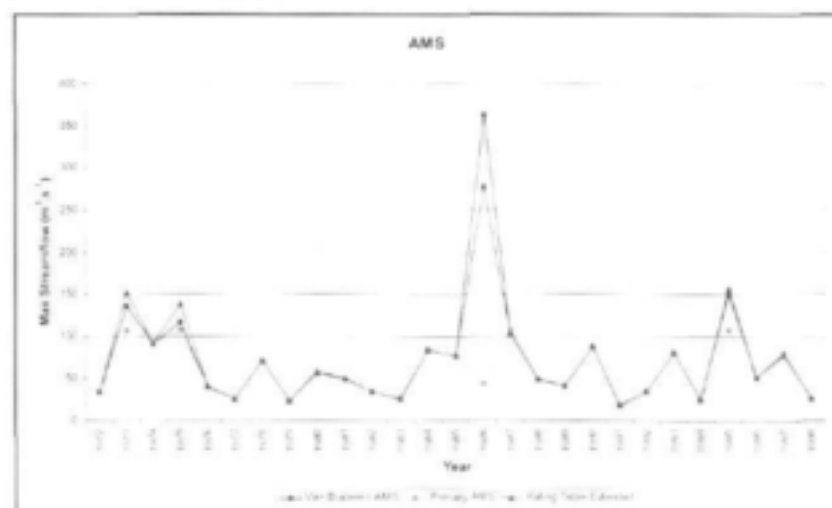


Figure 4.49 AMS at Gauging Weir V2H005

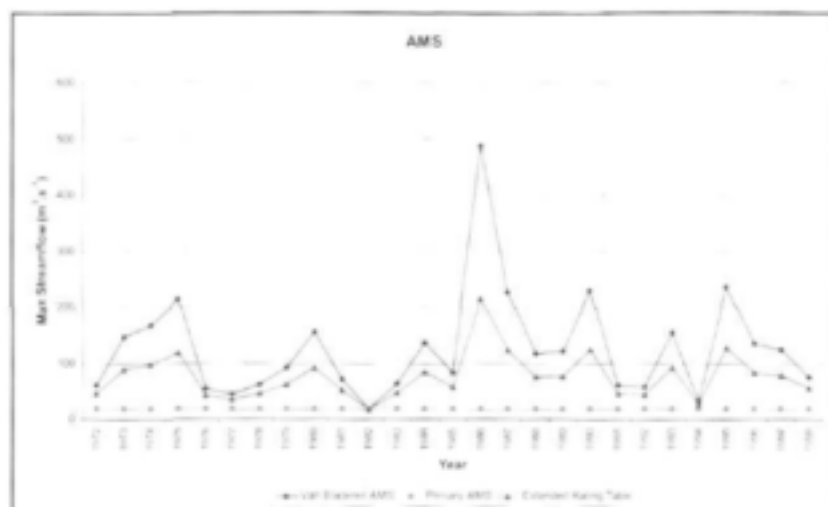


Figure 4.50 AMS at Gauging Weir V2H006

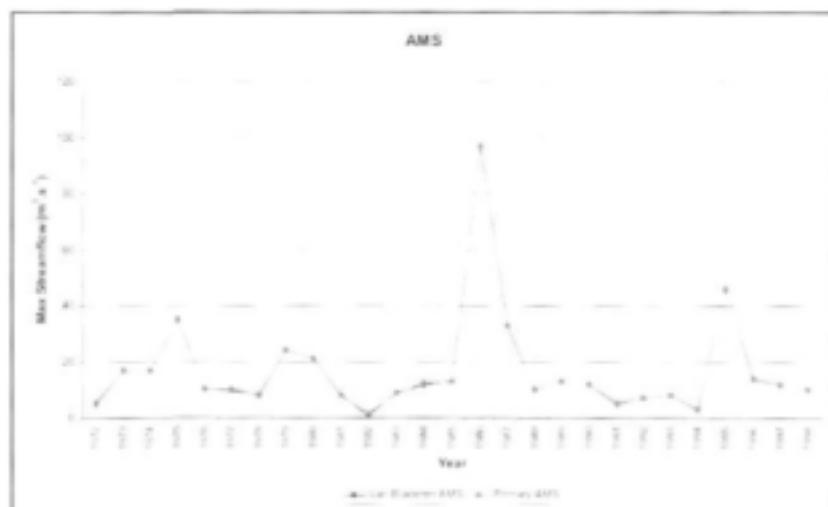


Figure 4.51 AMS at Gauging Weir V2H007

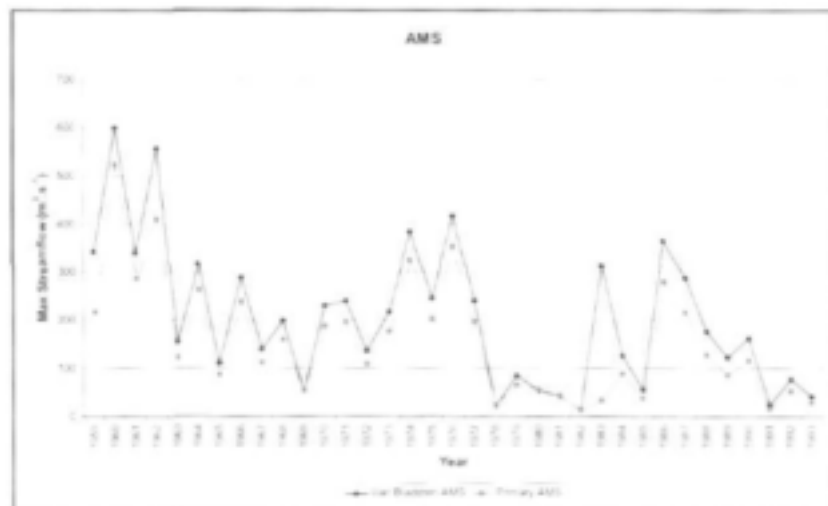


Figure 4.52 AMS at Gauging Weir V3H002

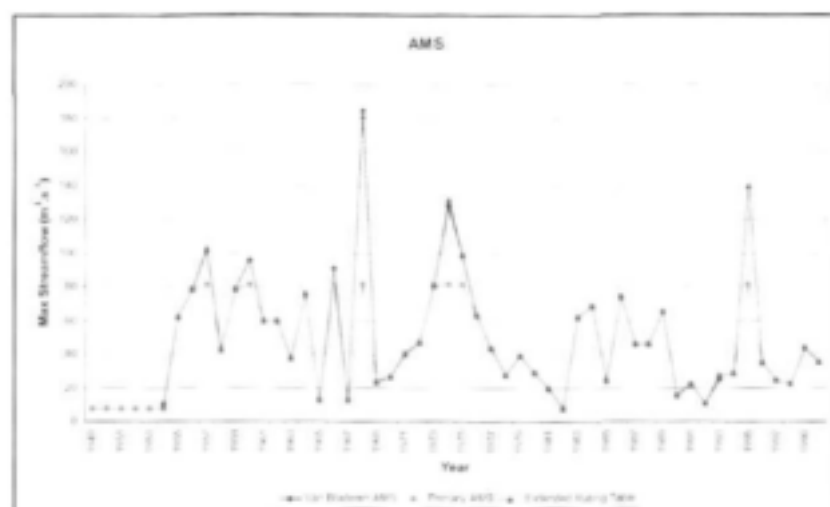


Figure 4.53 AMS at Gauging Weir V3H007

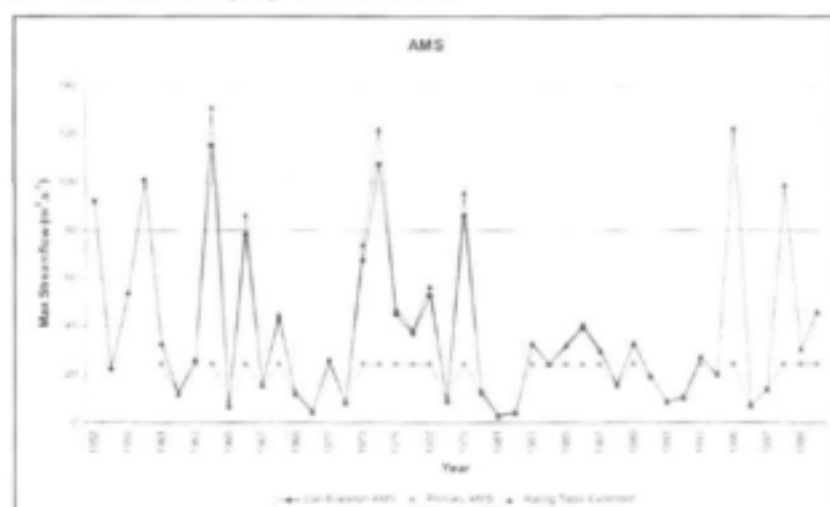


Figure 4.54 AMS at Gauging Weir V3H009

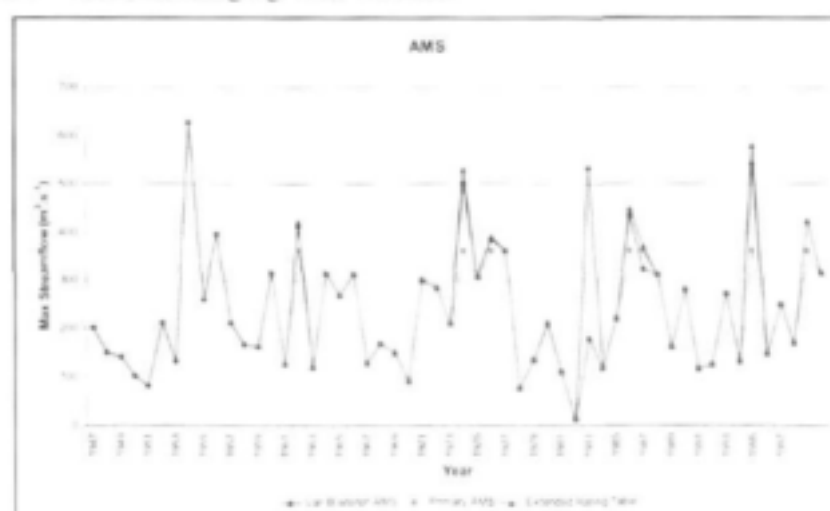


Figure 4.55 AMS at Gauging Weir V3H010

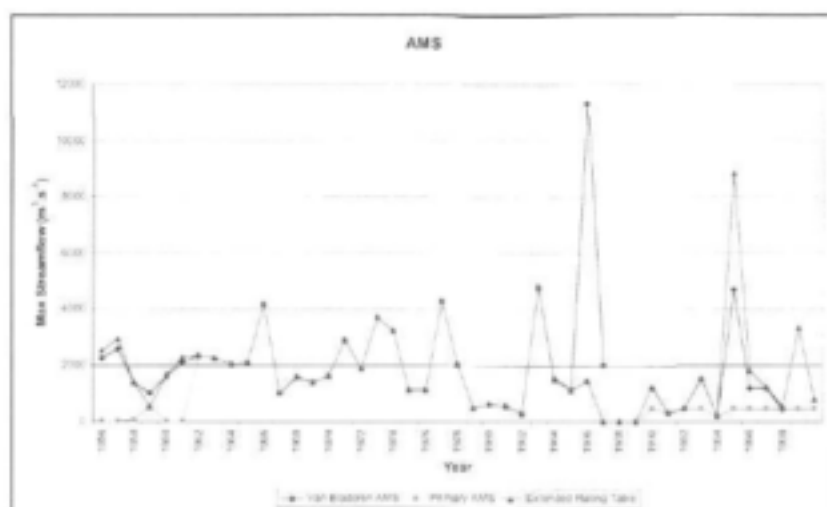


Figure 4.56 AMS at Gauging Weir V5H002

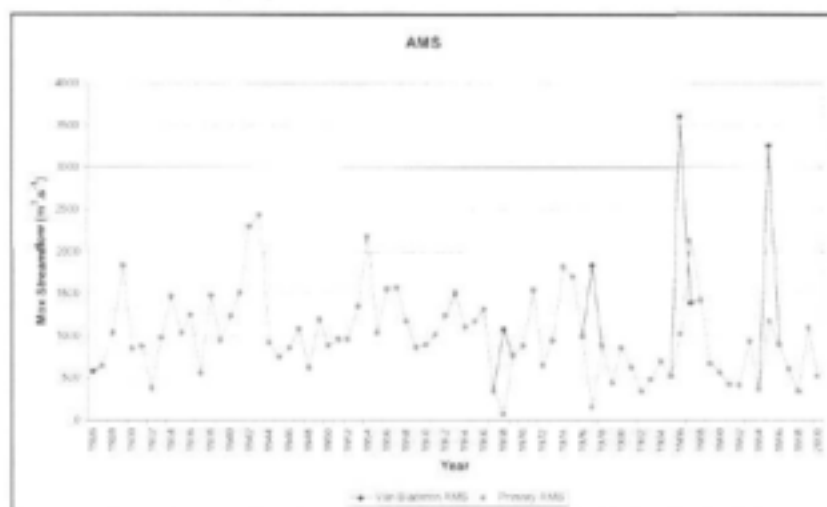


Figure 4.57 AMS at Gauging Weir V6H002

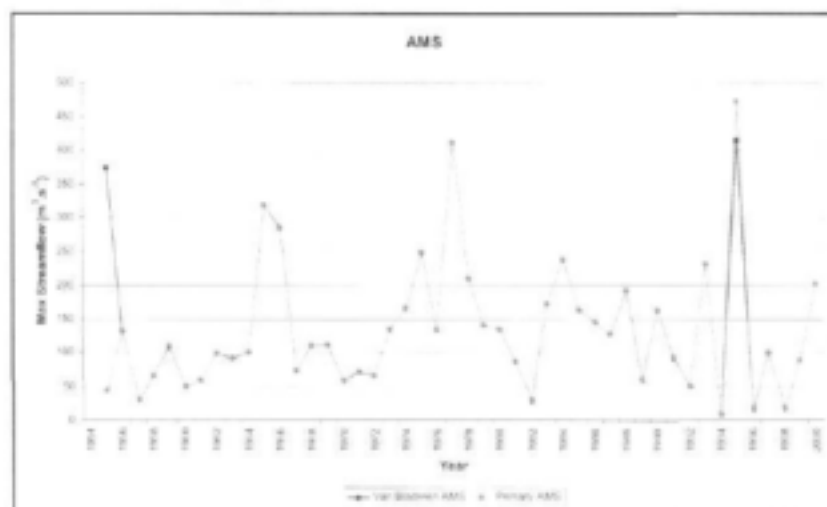


Figure 4.58 AMS at Gauging Weir V6H003

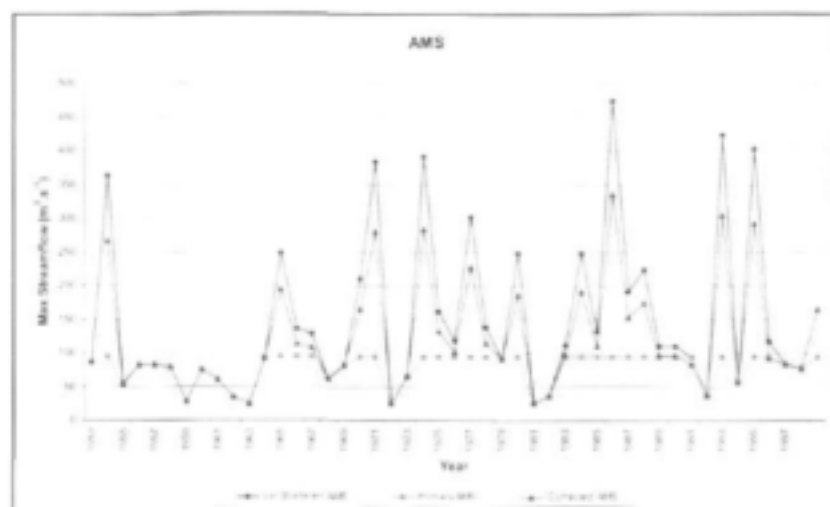


Figure 4.59 AMS at Gauging Weir V6H004

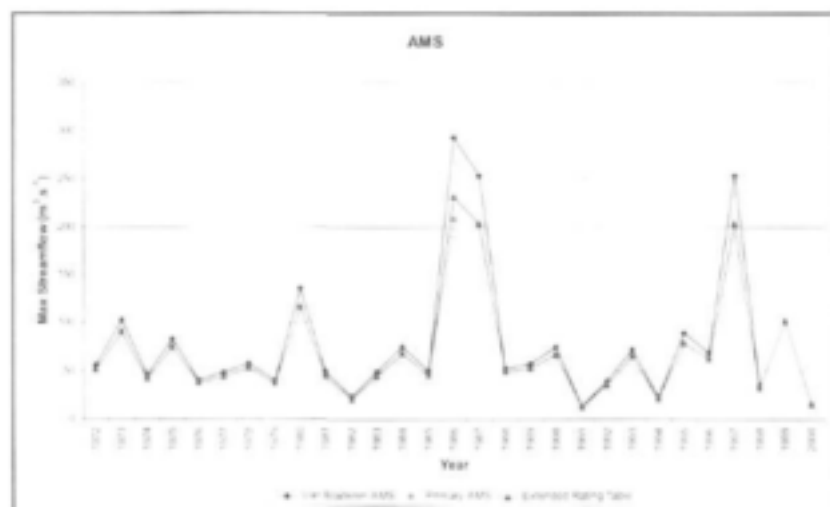


Figure 4.60 AMS at Gauging Weir V7H016

4.4.2. Examples of Estimated Design Floods

For the estimation of design floods, the maximum value in each year from both sources of AMS was used. As shown by the examples in

Figure 4.61 and Figure 4.62, the differences in the design floods estimated using the GEV and LP3 were generally not large and the LP3 distribution was adopted for design flood estimation in this study.

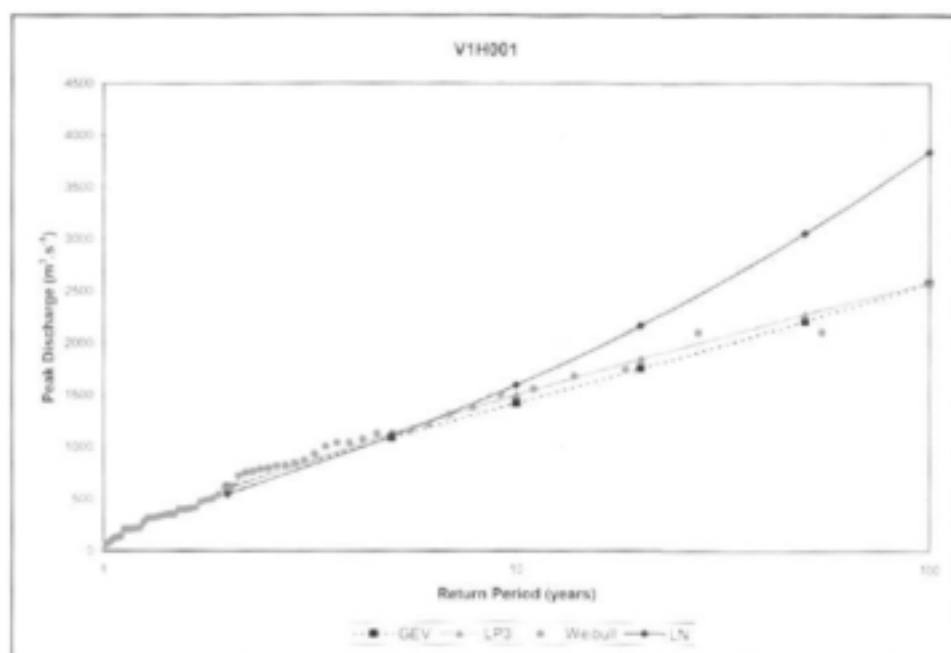


Figure 4.61 Estimated design floods at Gauging Weir V1H001

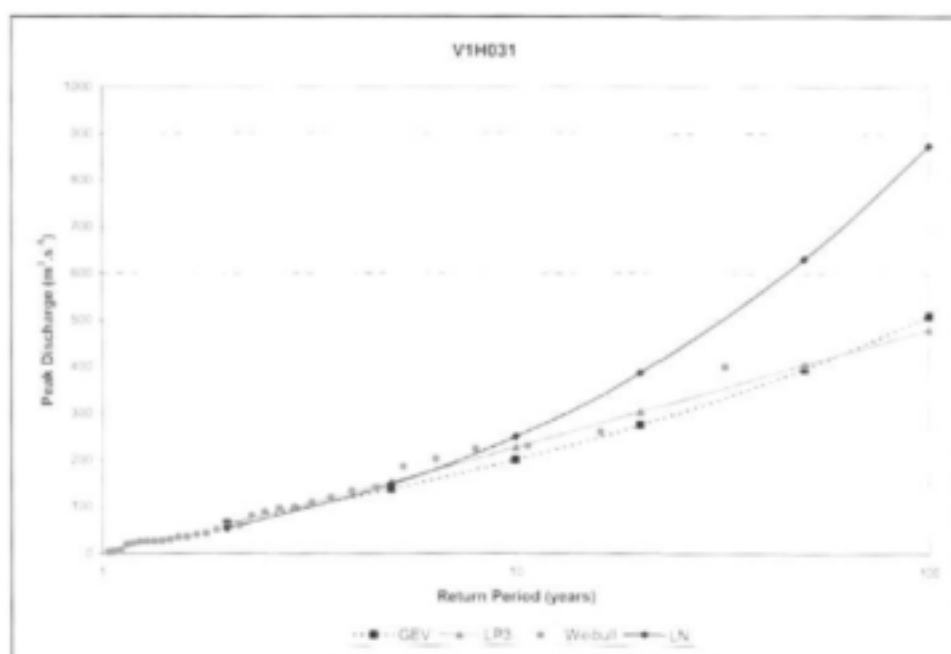


Figure 4.62 Estimated design floods at Gauging Weir V1H031

4.4.3. Influence of Rainfall Distribution on Design Floods

As shown in Section 3.3, a stochastic process was developed in this study to disaggregate the daily rainfall total into shorter durations when estimating peak discharge and flood routing in the *ACRU* model. Thus, a different temporal distribution of a rainfall event on a given day will be produced each time the model is run which will result in a different shape of the hydrograph and peak discharge. A simple illustration of this is shown in Figure 4.63 which contains the results from 3 simulation runs.

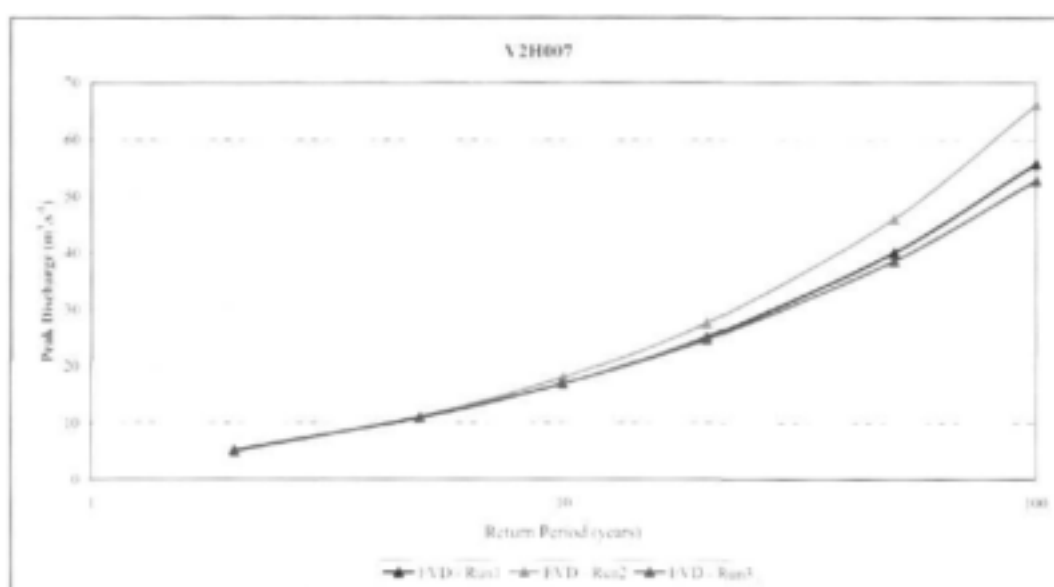


Figure 4.63 Example of the effect of rainfall disaggregation on design peak discharge

4.5. Verification of CSM

Verification of the simulated streamflow was performed at selected gauging weirs where the data were deemed to be of an acceptable quality and which did not have significant periods of missing data. The initial selection of gauges was guided by the weirs selected by Joubert and Hurley (1994). The locations of the gauging weirs used in the verification studies are shown in Figure 4.5. In the verification studies performed, a number of analyses were performed:

- Accumulated simulated and observed runoff is compared for periods when non-missing observed data are available and considered to be reliable.
- A comparison of frequency analyses of simulated and observed daily runoff depth and peak discharge is performed, with only non-missing values which are greater than zero included in the analyses
- Design floods computed from the simulated and observed AMS are compared.

4.5.1. Gauging Weir V1H009

As shown in Figure 4.64, the total streamflow simulated at V1H009 (196.3 km²) exceeds the observed total, but the rate of accumulation is similar. The impact of irrigation on the simulated results is also evident in Figure 4.64. The average annual

observed runoff : rainfall ratio for Catchment VIH009 is 11.5% and for an adjacent catchment which has a similar catchment area (V7H012), the ratio is 19.3%. Changes to the rating table were made in 1987 and 1988. It is interesting to note that up to 1987/1988 the accumulated simulated and observed runoff values diverged and start converging after this time. It is thus postulated that the apparent over-simulation may be the result of errors and changes in flow gauging in this catchment. The model under-simulated the streamflow for the period Dec 1995 to April 1996 when 686 mm of rainfall was recorded and the runoff : rainfall ratio for the simulated values is 19% and for the observed data 56%.

A frequency distribution of observed and simulated daily streamflow depth and peak discharge are shown in Figure 4.65 and Figure 4.66 respectively and which reflect the same trends shown in the accumulated daily flows with an over-simulation of the smaller events and under-simulation of the larger events. As shown in Figure 4.67, the design floods computed from the observed and simulated values are significantly smaller than the observed values. The impact of using the SCS lag equation, instead of the Schmidt-Schulze lag equation is also shown in Figure 4.67.

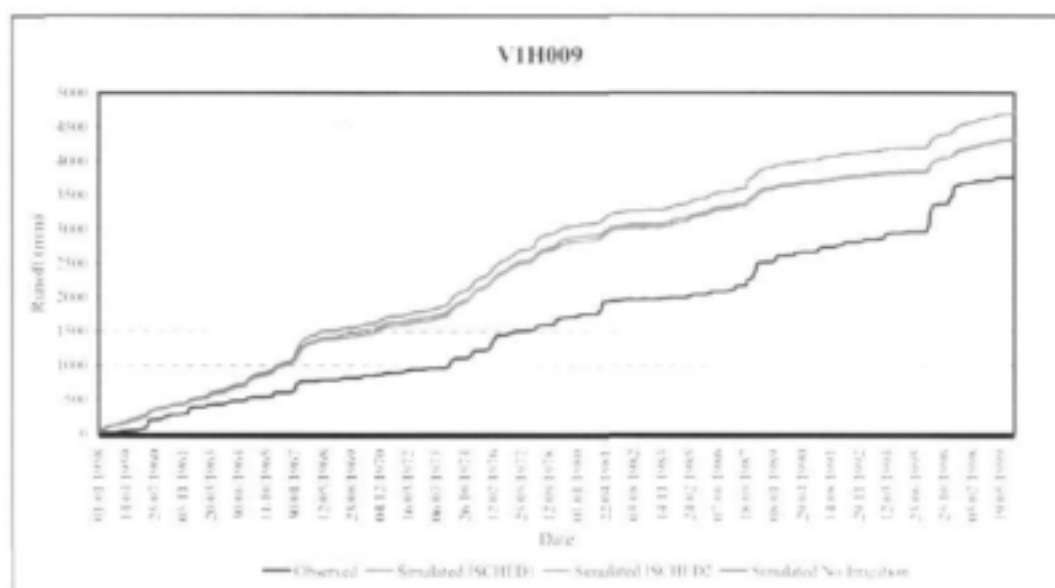


Figure 4.64 Accumulated observed and simulated flows at VIH009

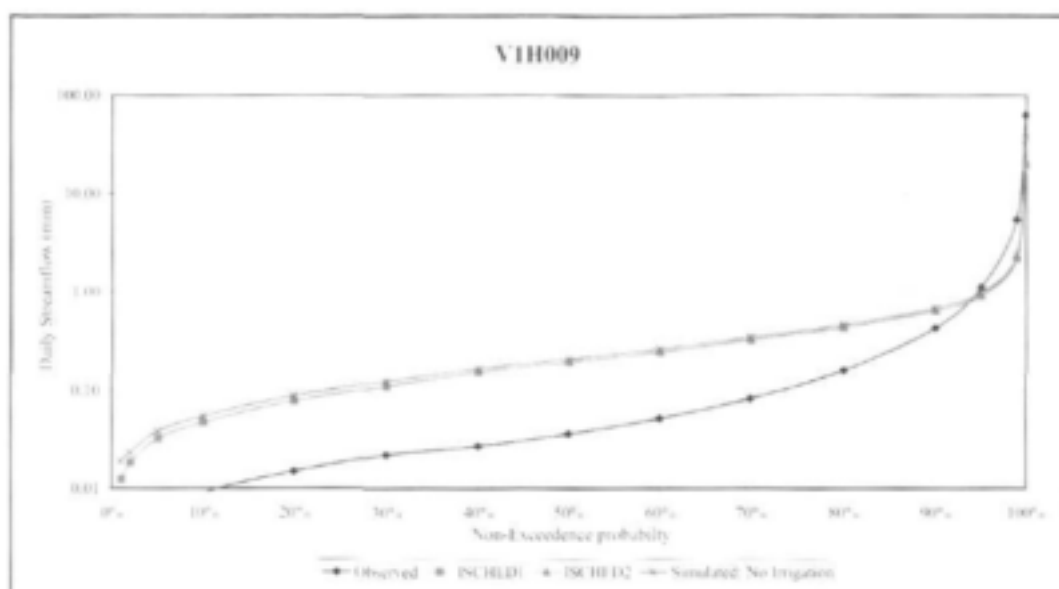


Figure 4.65 Frequency distribution of observed and simulated daily streamflow depth at V1H009

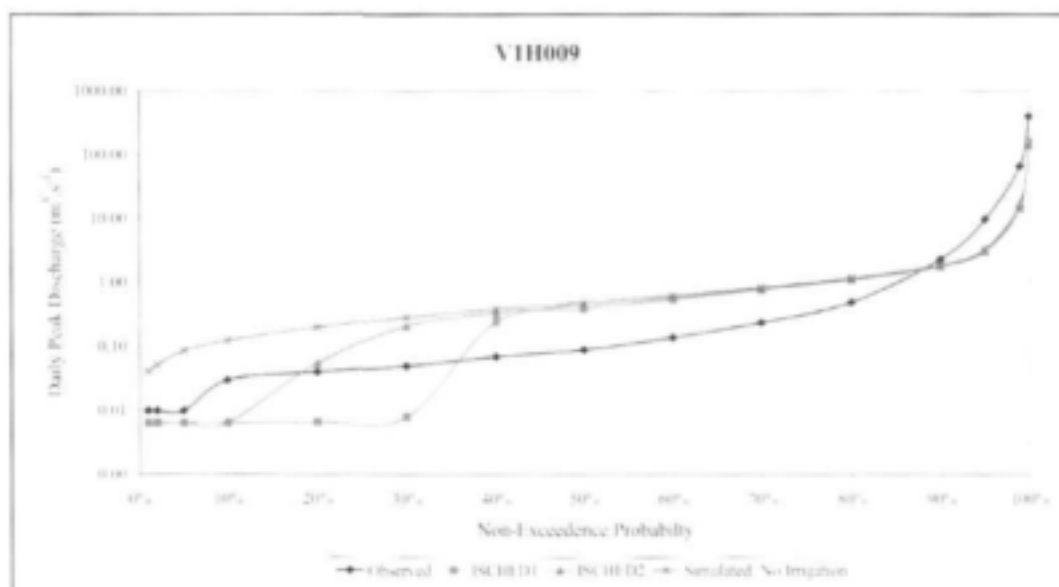


Figure 4.66 Frequency distribution of observed and simulated daily peak discharge at V1H009

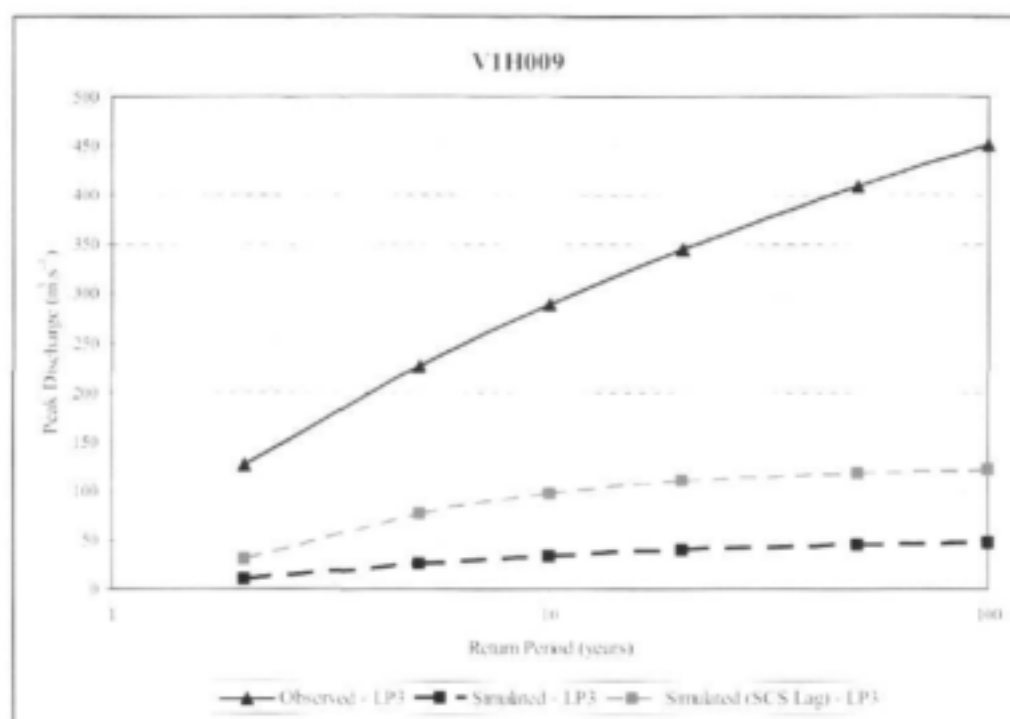


Figure 4.67 Design floods computed from the observed and simulated data at VIH009

4.5.2. Gauging Weir VIH010

As shown in Figure 4.68 the total streamflow simulated at VIH010 (781.1 km²) under estimates the observed total, but the agreement between the observed and simulated values improves towards the end of the record. This may be attributed to the land cover information used in the simulation and which reflects conditions in the catchment in 1999. The impact of irrigation on the simulated results is also evident in Figure 4.68. The average annual observed runoff : rainfall (Raingauge 0267693W) ratio is 39.4% while for adjacent catchments the ratio is 11.5% for VIH009 and 26.8% for VIH001. Hence the under-simulation may be the result of incorrect observed flow data, the use of raingauges in the simulation which do not adequately represent the rainfall in the catchment, or the land cover information used in the modelling. Transfers of water into the catchment may also explain the apparent under-simulation of runoff volumes.

A frequency distribution of observed and simulated daily streamflow depth and peak discharge are shown in

Figure 4.69 and Figure 4.70 respectively and which reflect the under-simulation evident in the accumulated daily flows. Design floods computed from the observed and simulated values are shown in Figure 4.71 and the simulated values are significantly smaller than the observed values.

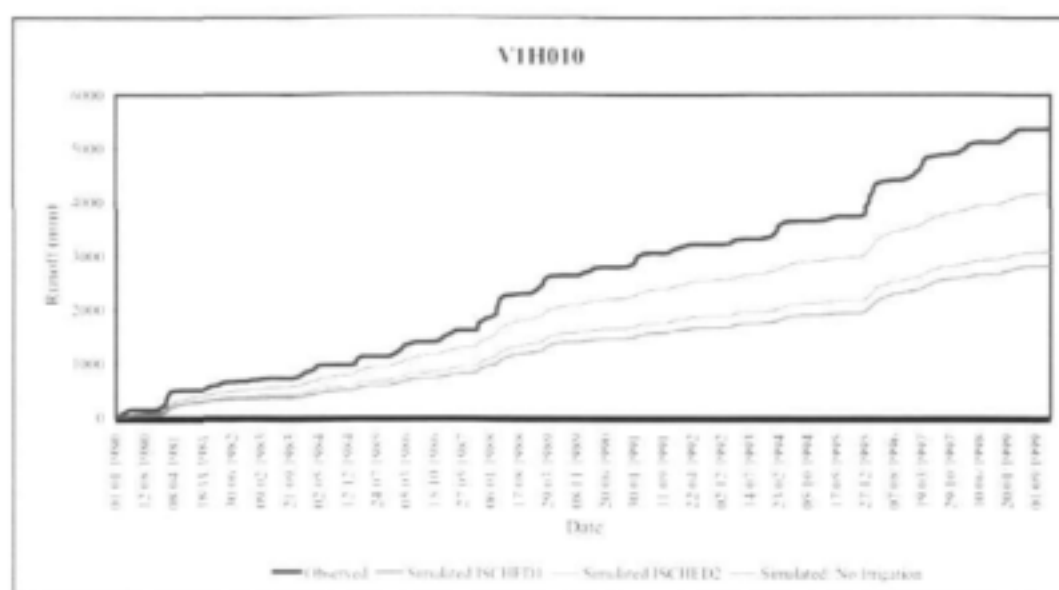


Figure 4.68 Accumulated observed and simulated flows at VIH010

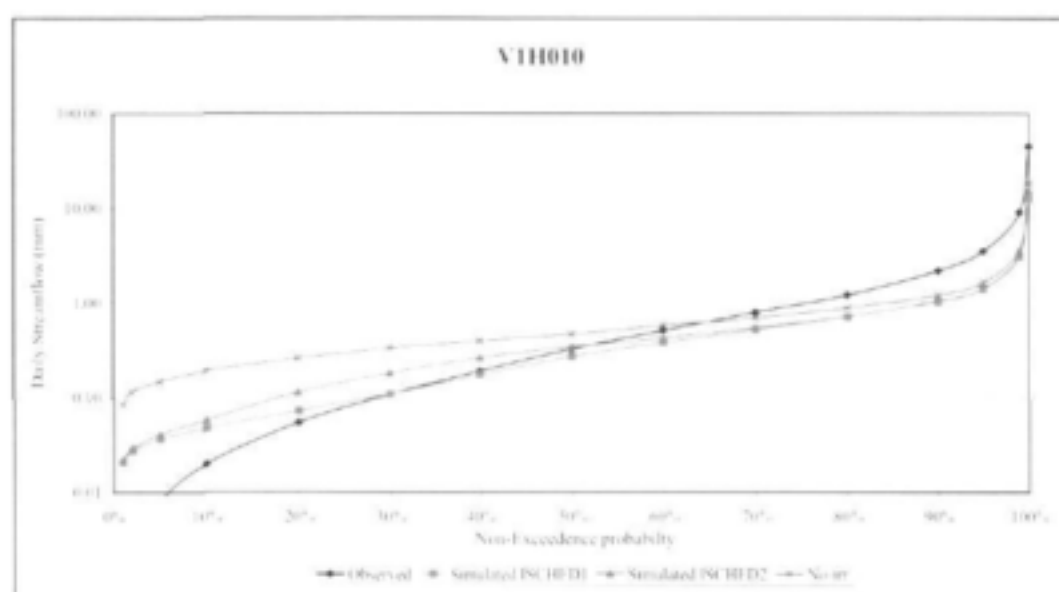


Figure 4.69 Frequency distribution of observed and simulated daily streamflow depth at VIH010

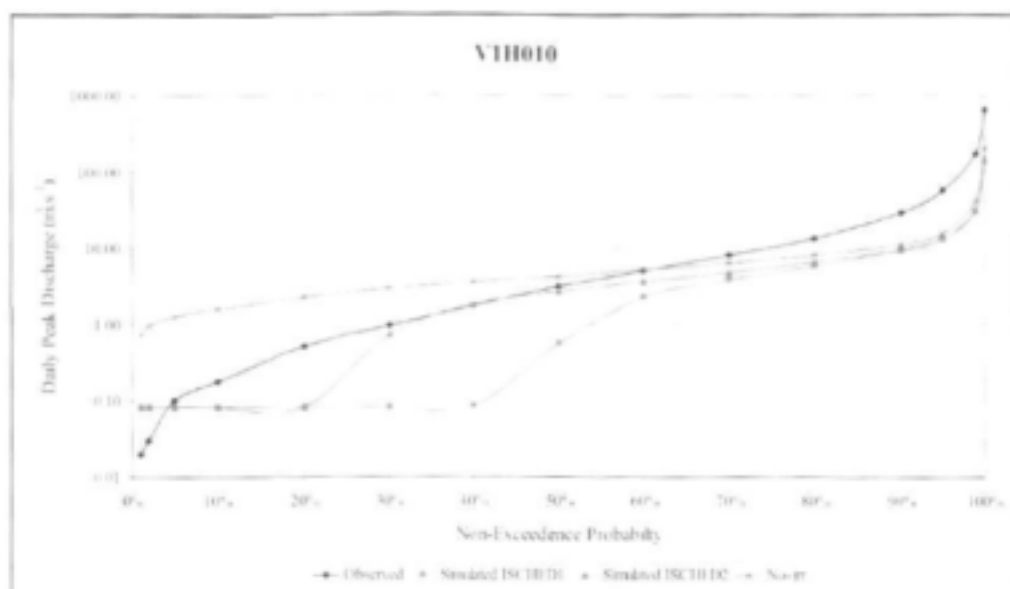


Figure 4.70 Frequency distribution of observed and simulated daily peak discharge at VIH010

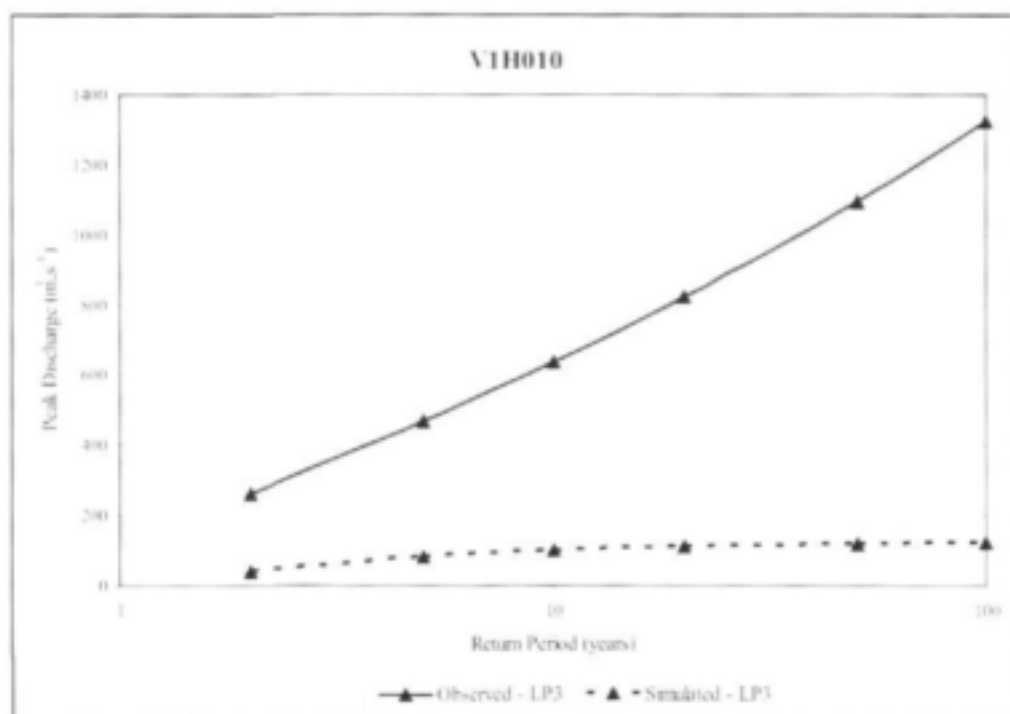


Figure 4.71 Design floods computed from the observed and simulated data at VIH010

4.5.3. Gauging Weir VIH026

As shown in Figure 4.72, the total streamflow simulated at VIH026 (1916.5 km²) compares well with the observed values until 1980, after which the trend in the observed data indicates that transfers of water upstream of the gauging weir occurred.

The frequency distribution of observed and simulated daily streamflow depth and peak discharge shown in Figure 4.73 and Figure 4.74 respectively are for the period from 1967 to 1980 indicate that both the larger daily streamflow volumes and peak discharges were under-simulated and smaller events over-simulated. Hence, the design floods computed from the observed and simulated values for the period from 1967 to 1980, shown in Figure 4.75, indicate under-simulation by the model.

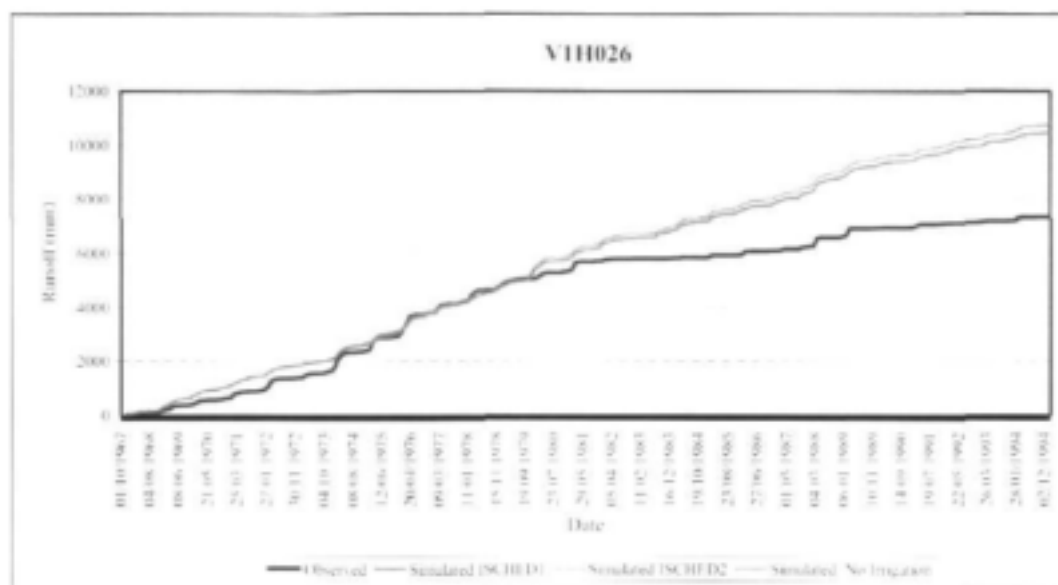


Figure 4.72 Accumulated observed and simulated flows at V1H026

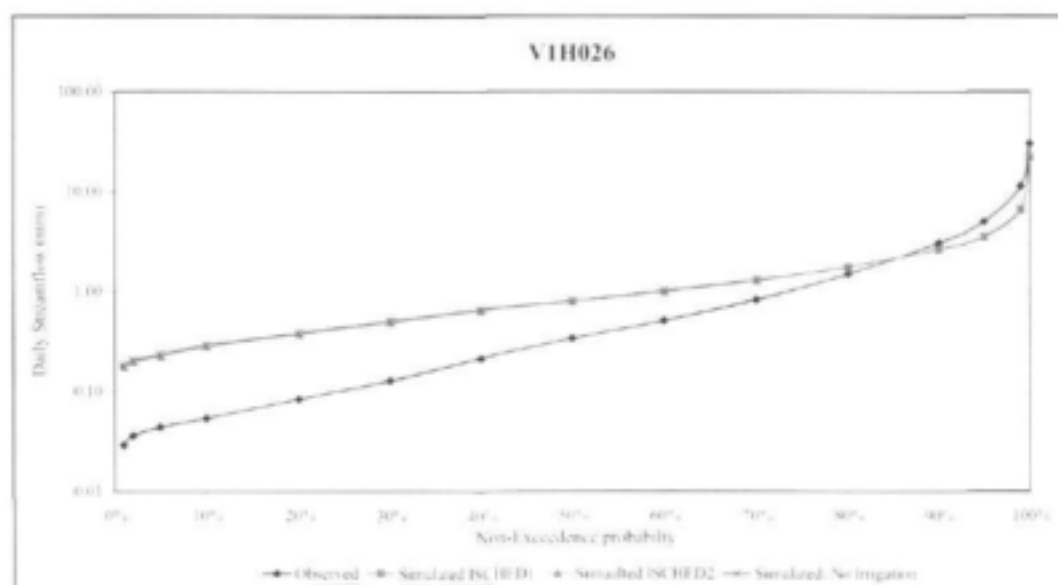


Figure 4.73 Frequency distribution of observed and simulated daily streamflow depth at V1H026

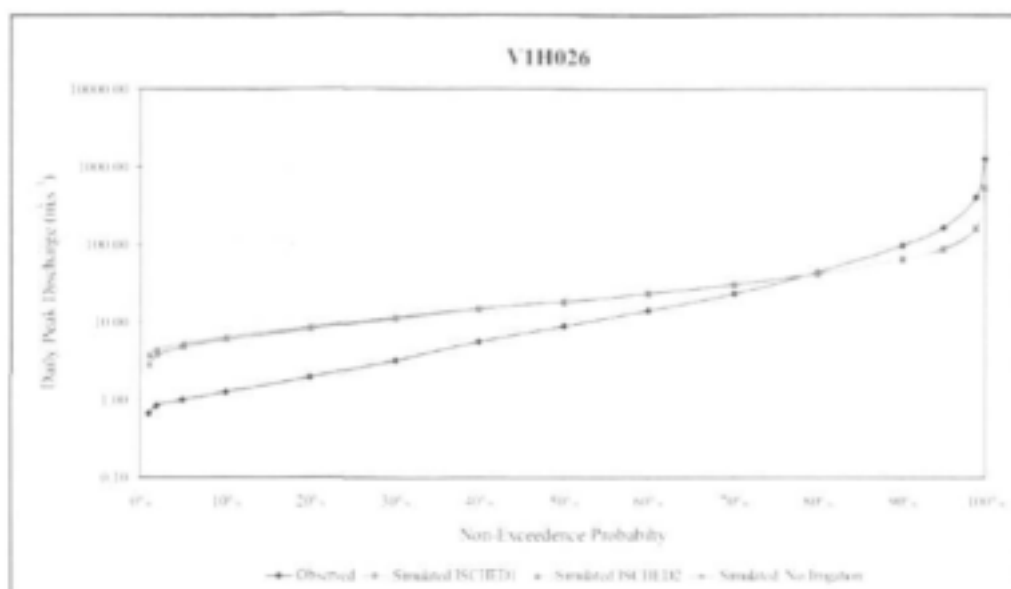


Figure 4.74 Frequency distribution of observed and simulated daily peak discharge at VIH026

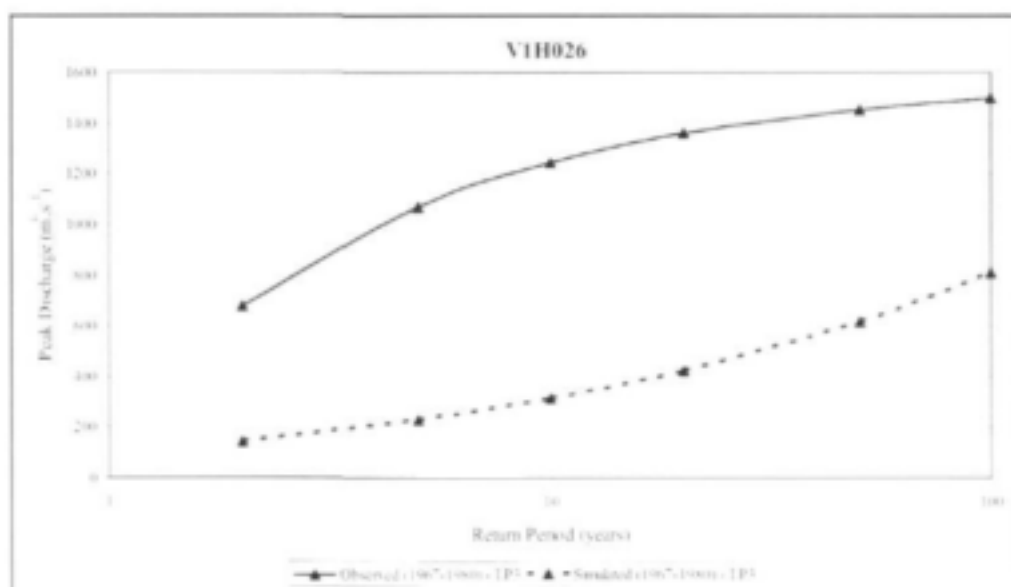


Figure 4.75 Design floods computed from the observed and simulated data at VIH026

4.5.4. Gauging Weir VIH031

As shown in Figure 4.76 the total streamflow simulated at VIH031 (163.4 km²), which includes irrigation, under-simulates the observed values for the first period of record (1972-1985) and over-simulates the observed values for the remainder of the record (1986-1999). The simulation without irrigation corresponds well with the observed values from 1972 to 1980 and exceeds the observed values after this point. Hence the deviation between the accumulated observed and simulated runoff may be attributed to the development of irrigation within the catchment from 1980 onwards.

The changing relationship between the observed runoff and rainfall shown in Figure 4.77 reflects the changing hydrological response from the catchment over time which could possibly be the result of increased irrigation in the catchment. While the total volume simulated appears to correspond well the observed total, the response to larger periods of rain in the simulated values is muted compared to the observed response.

A frequency distribution of observed and simulated daily streamflow depth and peak discharge are shown in Figure 4.78 and Figure 4.79 respectively and which indicate that the larger events volumes are simulated well, but the peak discharge is under-simulated. As a consequence, the design floods computed from the simulated values generally under-simulate the design floods computed from the observed data, as shown in Figure 4.80. Included in Figure 4.80 are design floods computed using the combined AMS extracted from the primary flow data and van Bladeren (2000) AMS (Primary&VB), and using the AMS extracted from the primary flow data only (Primary). The design peak discharges appear to be reasonable when compared to the design values computed using the primary data only.

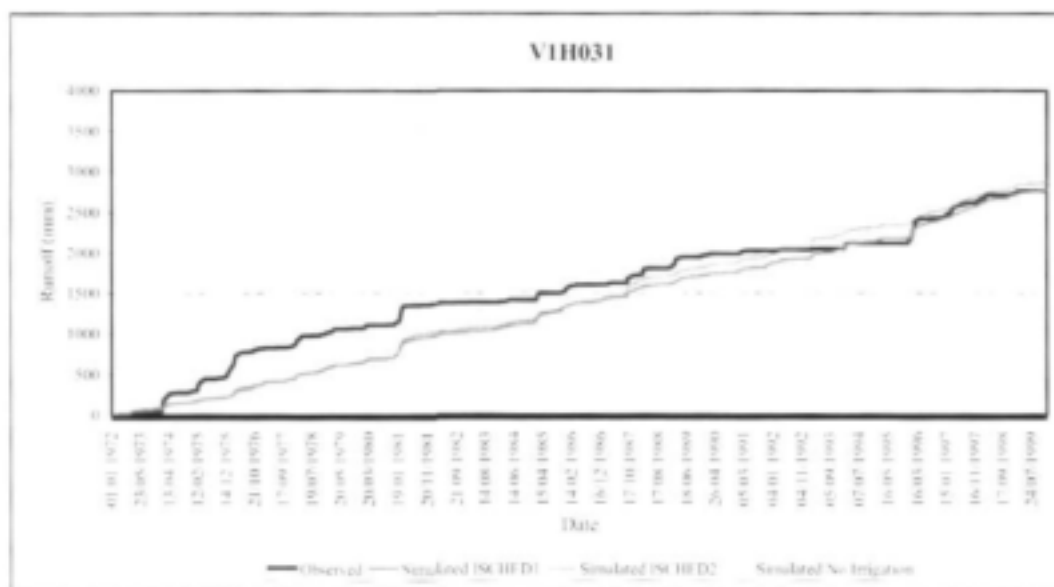


Figure 4.76 Accumulated observed and simulated flows at VIH031

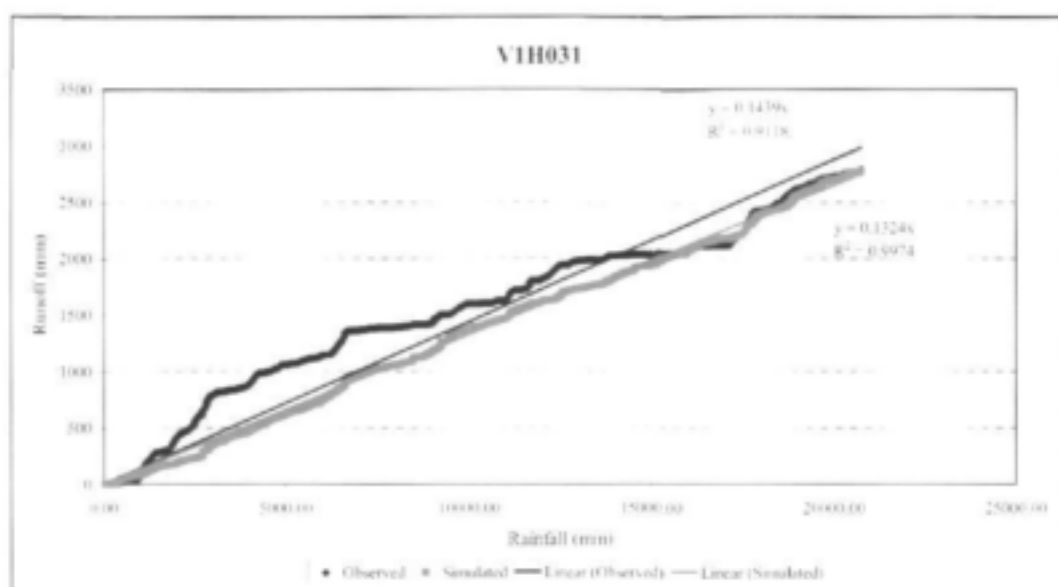


Figure 4.77 Accumulated runoff vs accumulated rainfall at V1H031

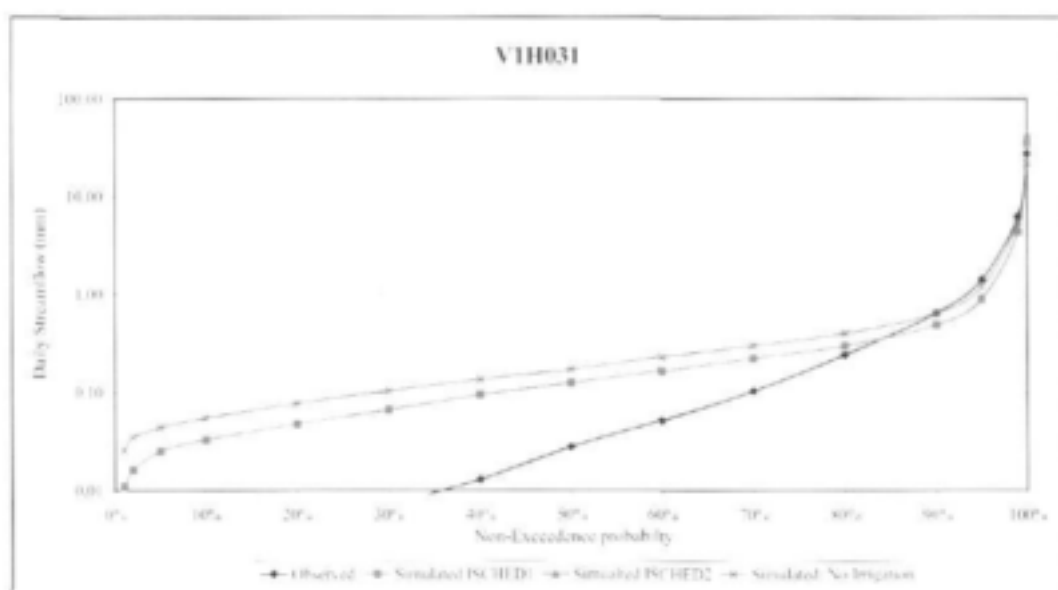


Figure 4.78 Frequency distribution of observed and simulated daily streamflow depth at V1H031

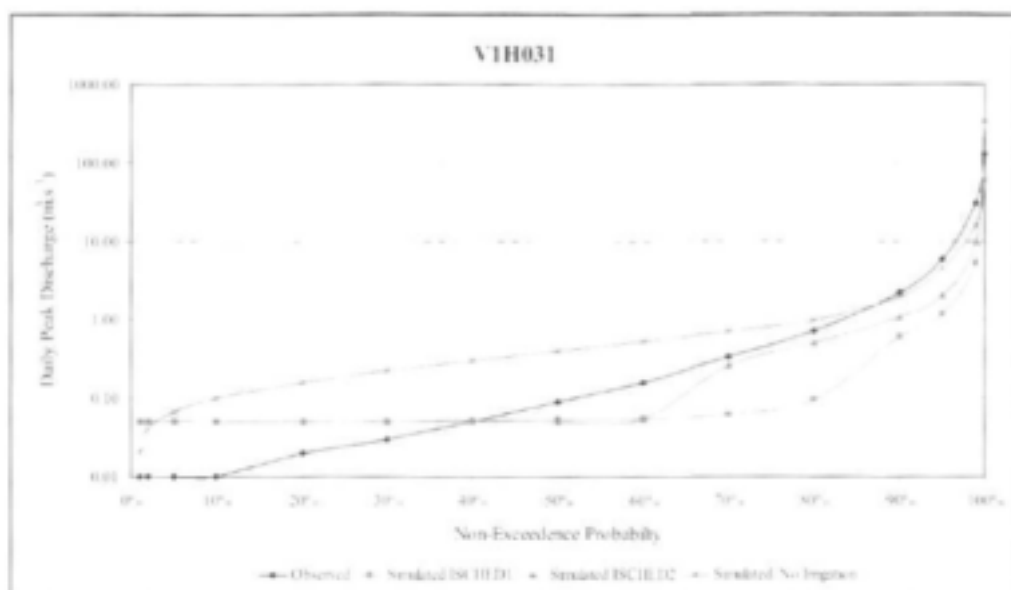


Figure 4.79 Frequency distribution of observed and simulated daily peak discharge at VIH031

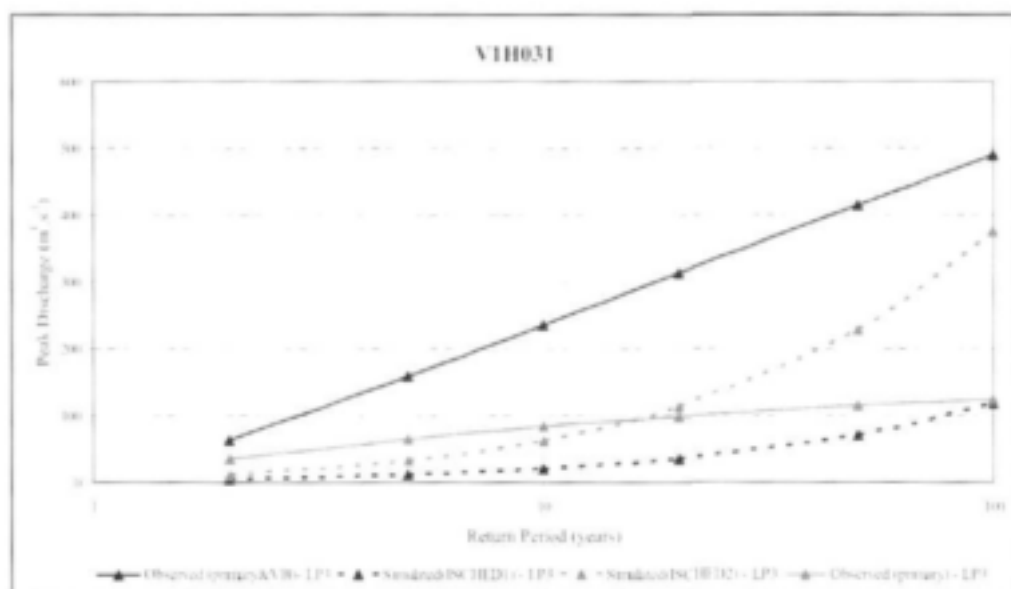


Figure 4.80 Design floods computed from the observed and simulated data at VIH031

4.5.5. Gauging Weir VIH038

As shown in Figure 4.81, the two irrigation scenarios result in similar simulated streamflow depths at VIH038 (1649.0 km²), both of which correspond extremely well with the observed values up to 1980, after which the simulated volumes exceed the observed values up to approximately 1990, after which there is good agreement between the simulated and observed values. One explanation of the improved simulations towards the end of the period is that the land cover information used in the simulation was captured in 1999 and hence the simulations are expected to be

realistic around 1999. A frequency distribution of the observed and simulated daily streamflow depths and peak discharges are shown in Figure 4.82 and Figure 4.83 respectively. The distribution of the larger observed event volumes is well represented in the simulated values. However, the distribution of larger daily peak discharges is under-simulated which results in the under-simulation of design floods, as shown in Figure 4.84.

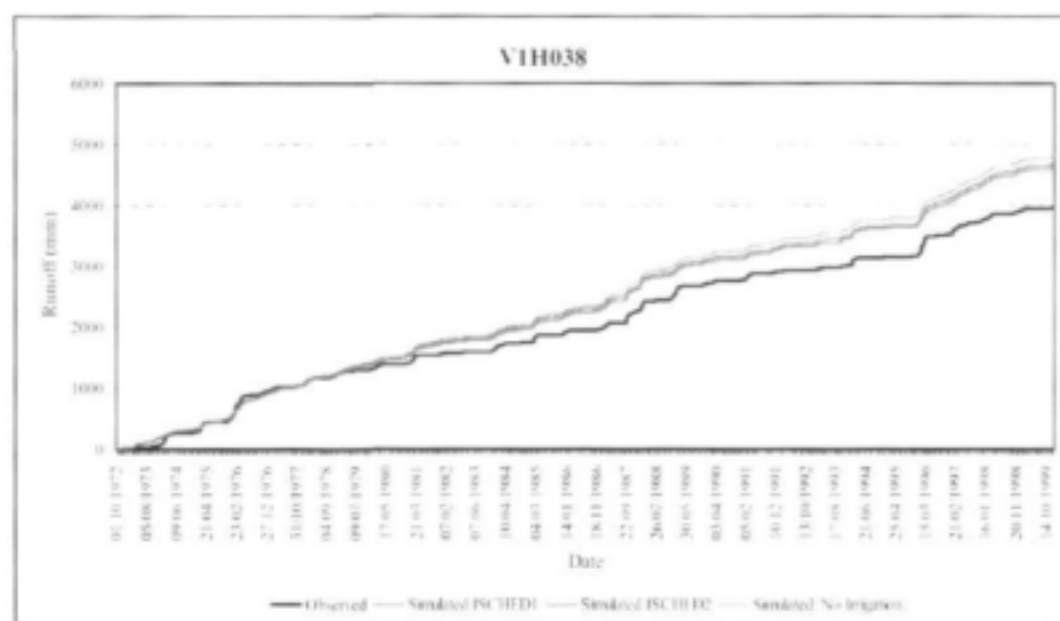


Figure 4.81 Accumulated observed and simulated flows at V1H038

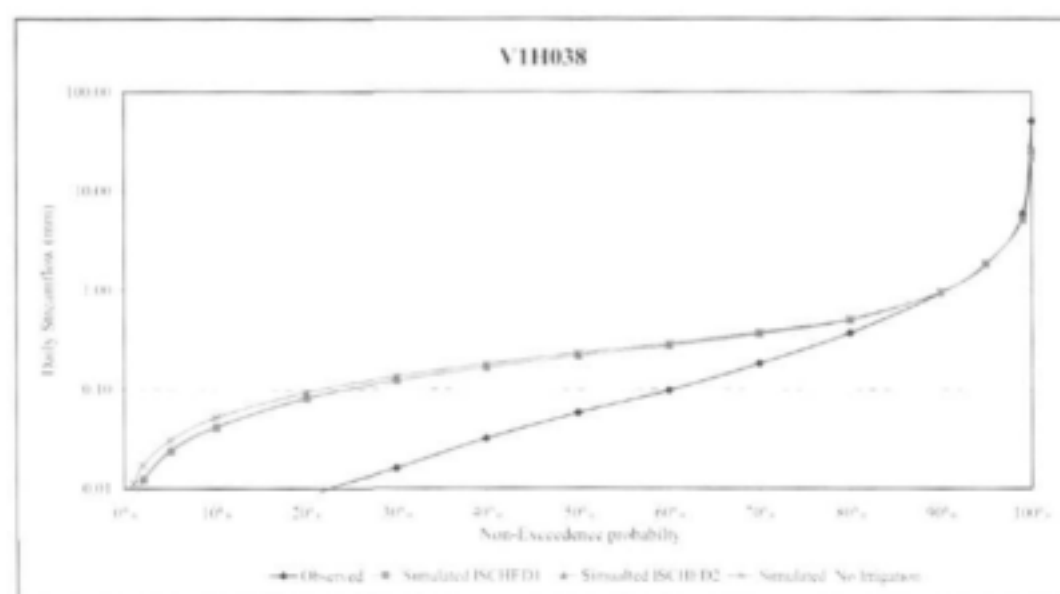


Figure 4.82 Frequency distribution of observed and simulated daily streamflow depth at V1H038

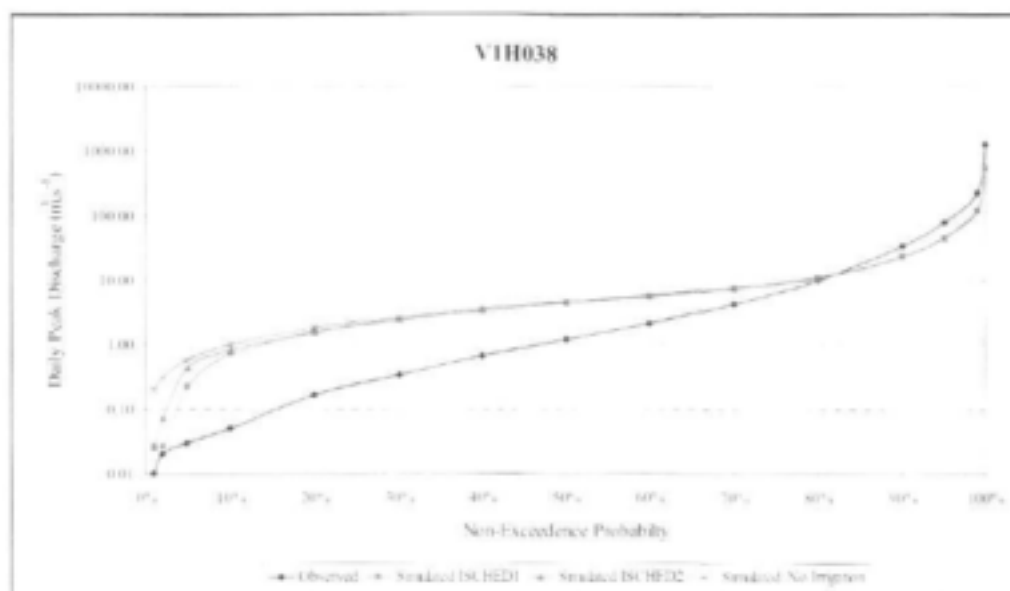


Figure 4.83 Frequency distribution of observed and simulated daily peak discharge at V1H038

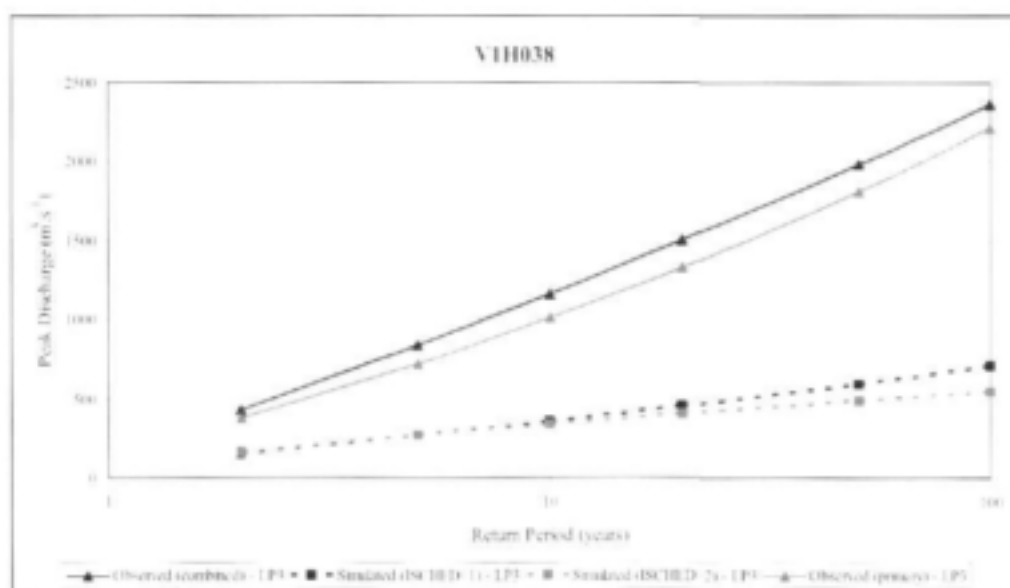


Figure 4.84 Design floods computed from the observed and simulated data at V1H038

4.5.6. Gauging Weir V2H007

As shown in Figure 4.23 the relationship between the observed runoff data at Weir V2H007 appears to be relatively consistent at an annual level with rainfall measured at Raingauge 0268352W. Relatively little flow data was missing in the observed record and 18% of data missing in the 1983 hydrological year occurred in the winter months. Hence the entire period of observed record was used in the verification study.

The accumulated observed and simulated runoff at Gauging Weir V2H007 (111.3 km²) for the period of available observed flow data (1972-2000) is shown in Figure 4.85 and frequency analyses of daily runoff depths and daily peak discharges are shown in Figure 4.86 and Figure 4.87 respectively. These results indicate that the CSM, including the simulation of irrigation, simulated the observed runoff volume reasonably well and there was little difference between the two irrigation scheduling scenarios considered. The influence of irrigation in the catchment is shown by the results which did not simulate irrigation and the development of irrigation in the catchment over time is evident by the differences in the observed and simulated values for the simulation with no irrigation. The simulated results generally agree better with the observed values for the latter part of the record when the land cover used in the simulation reflects the 1999 land cover in the catchment. As shown in Figure 4.87 the frequency distribution of daily peak discharge performed on the simulated and observed values are very similar, over the entire range of flows. Similarly, as shown in Figure 4.88, the design floods computed from the observed and simulated values agree closely.

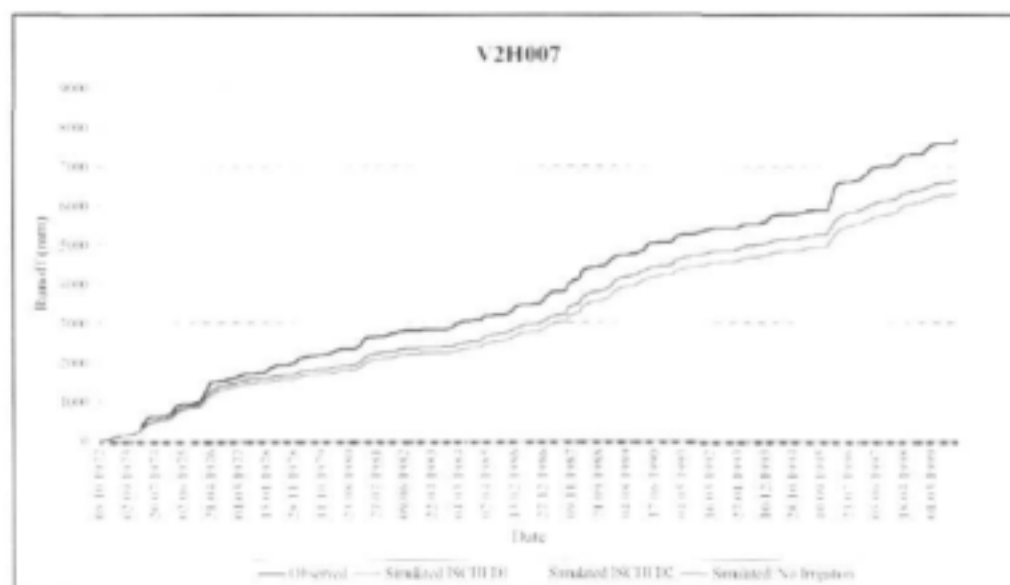


Figure 4.85 Accumulated simulated and observed runoff at Gauging Weir V2H007

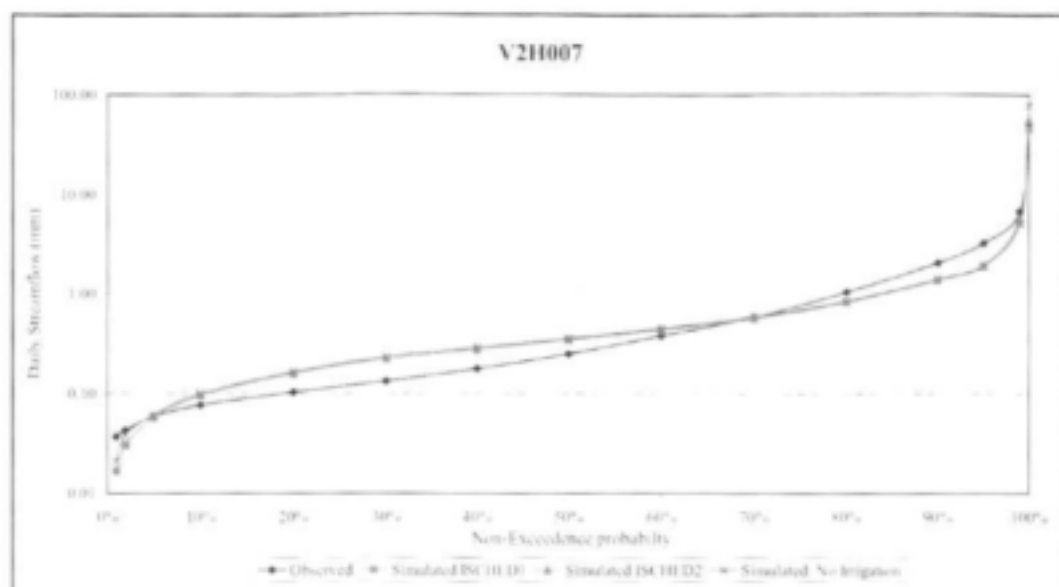


Figure 4.86 Frequency distribution of observed and simulated daily streamflow depth at V2H007

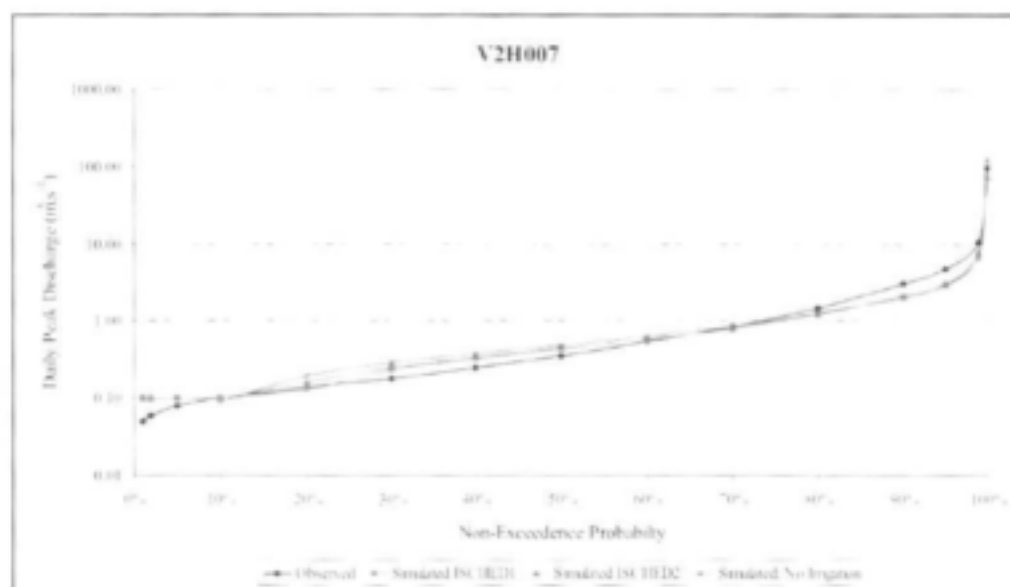


Figure 4.87 Frequency distribution of observed and simulated daily peak discharge at V2H007

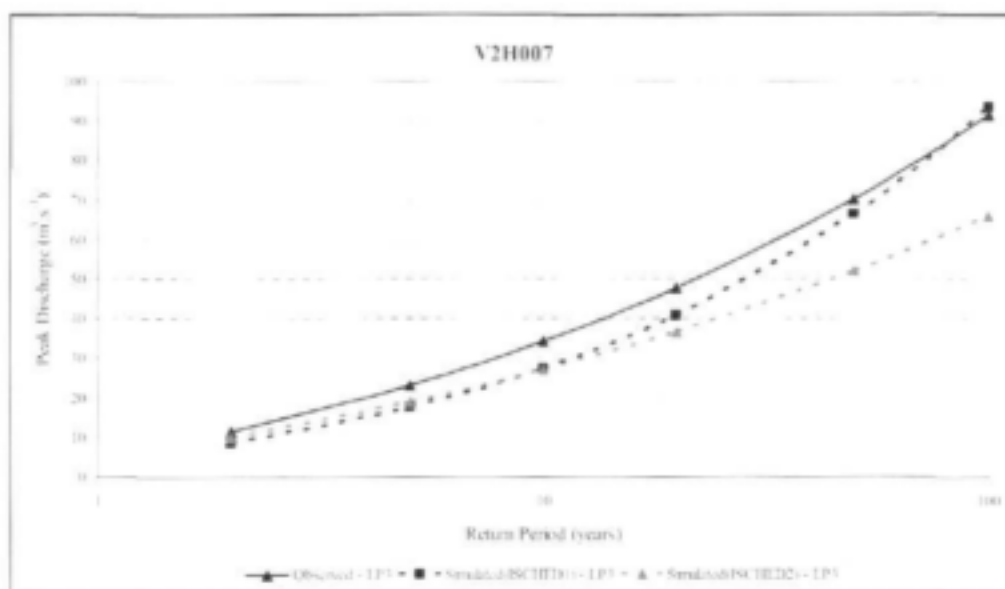


Figure 4.88 Design floods computed from the observed and simulated data at V2H007

4.5.7. Gauging Weir V2H016

The accumulated simulated and observed runoff at V2H016 (154.3 km²) for the period of available observed flow data (1983-2000) is shown in Figure 4.89 and a frequency analysis of daily runoff depths is shown in Figure 4.90. These results indicate that the CSM generally simulated the observed runoff volume well at V2H016, except for the event on 1-3 March 1988 where the rainguage used in the simulation (0269611W) reported 70, 29 and 100 mm of rainfall over this period. A nearby rainguage (0269532A) reported 20, 25 and 14 mm over the corresponding period. Hence the over-simulation for this period is probably caused by the rainfall data used in the simulation.

As shown in Figure 4.91, the frequency distribution of daily peak discharge performed on the simulated and observed values are similar, but the observed daily peak discharges are over-simulated at higher flows. As a consequence, the design floods computed from the observed are over-simulated, as shown in Figure 4.92.

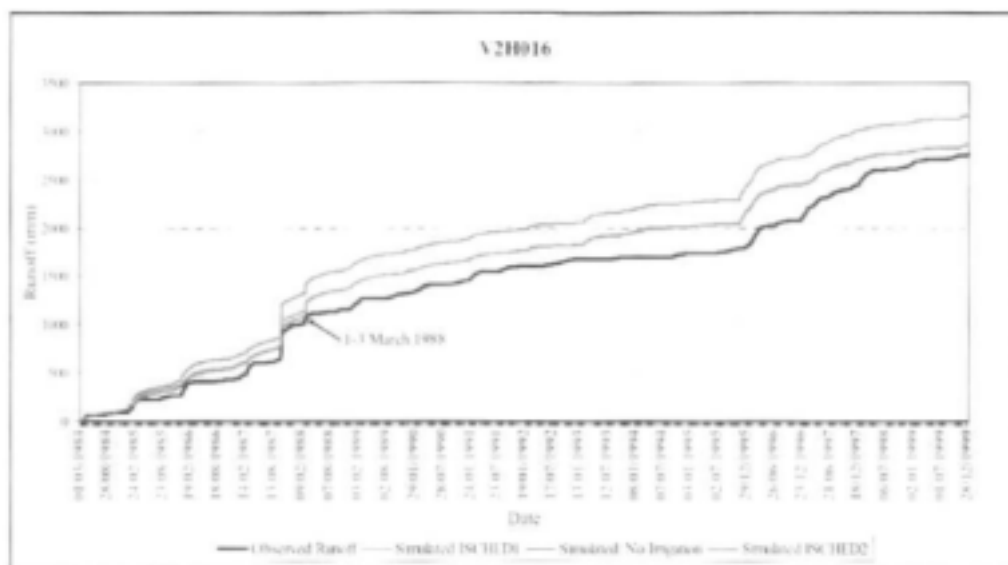


Figure 4.89 Accumulated simulated and observed runoff at Gauging Weir V2H016

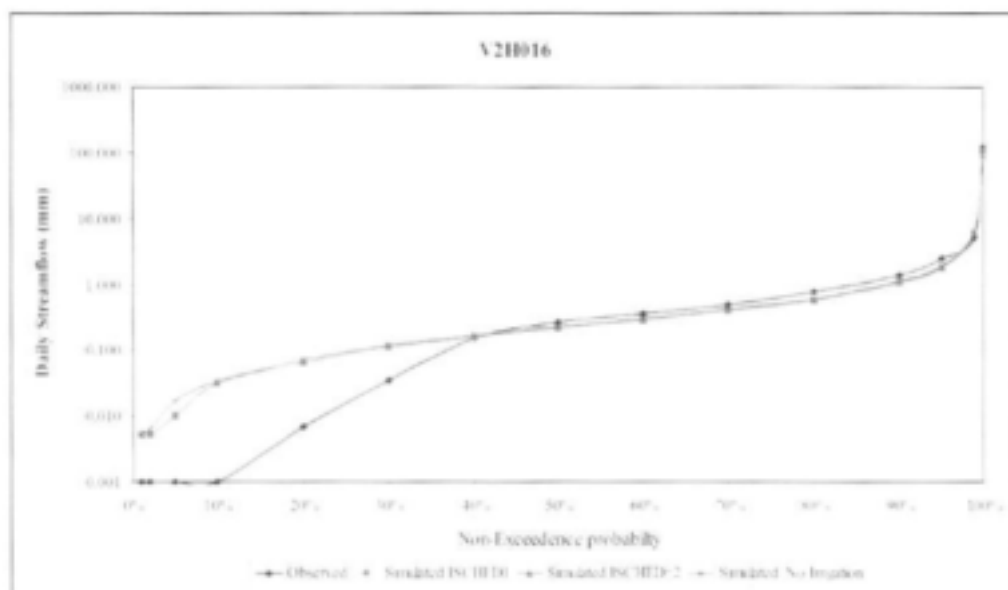


Figure 4.90 Frequency analysis of simulated and observed daily runoff depth at Gauging Weir V2H016

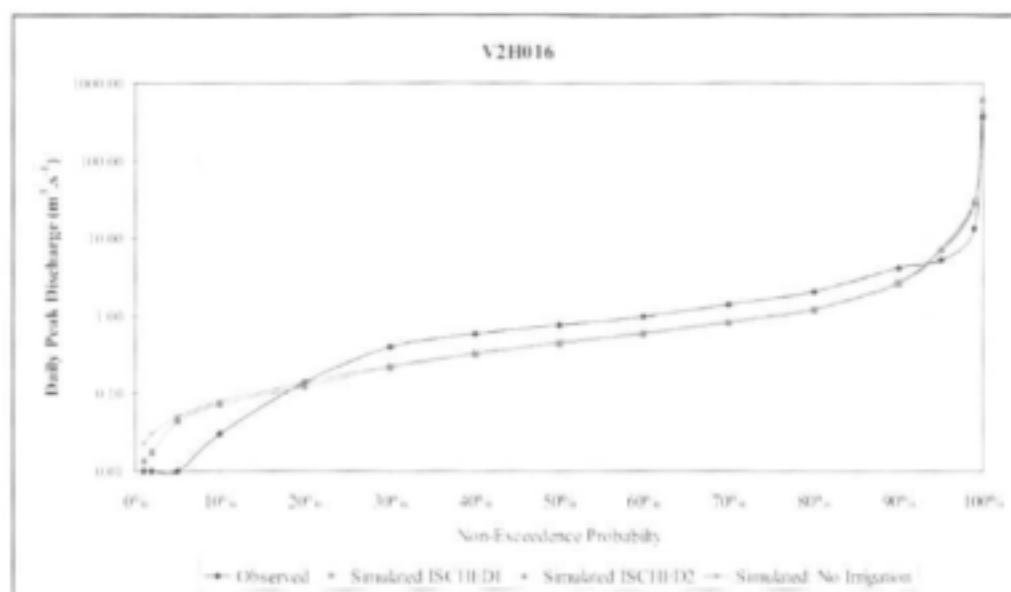


Figure 4.91 Frequency analysis of simulated and observed daily peak discharges at Gauging Weir V2H016

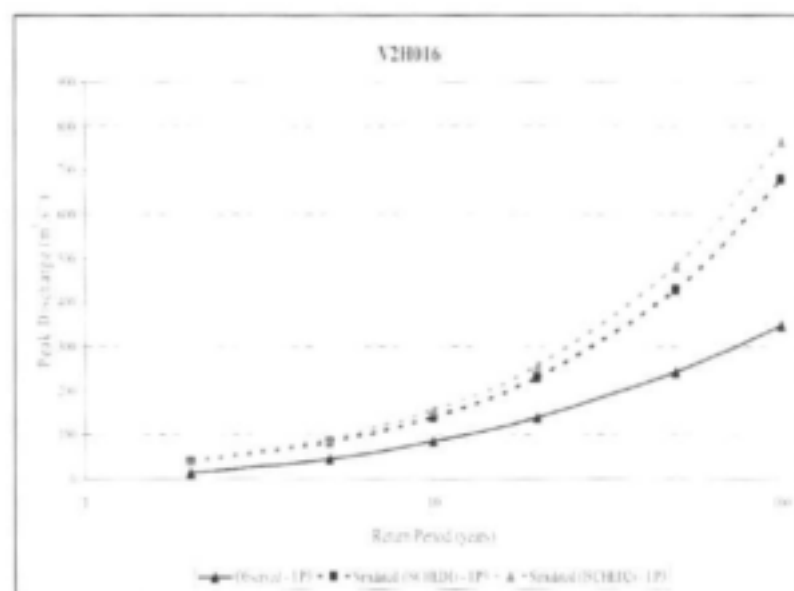


Figure 4.92 Design floods computed from the observed and simulated data at V2H016

4.5.8. Gauging Weir V3H007

The accumulated simulated and observed runoff at V3H007 (126.9 km²) for the period of available reliable observed flow data is shown in Figure 4.93 and a frequency analysis of daily runoff depth is shown in Figure 4.94. These results indicate that the CSM generally under-simulated the observed runoff volume prior to 1976, but followed the trends in the observed data well after 1976. The land cover information used was captured in 1999 and hence it is postulated that these trends in the simulated volume may be attributed to developments in the catchment and

changes to the land cover in the catchment over the period considered, i.e. the simulations are expected to be better for the period around 1999 when the land cover used in the simulations reflect actual conditions in the catchment. The influence of irrigation on runoff volumes in this catchment is not significant.

As shown in Figure 4.95, the frequency distribution of daily peak discharge performed on the simulated and observed data are similar, but the simulated daily peak discharges exceed the observed values at higher flow rates. The design floods computed from the observed and simulated data reflect this trend in the peak discharges with design floods computed from the simulated values exceeding the design flood computed from the observed data, as shown in Figure 4.96.

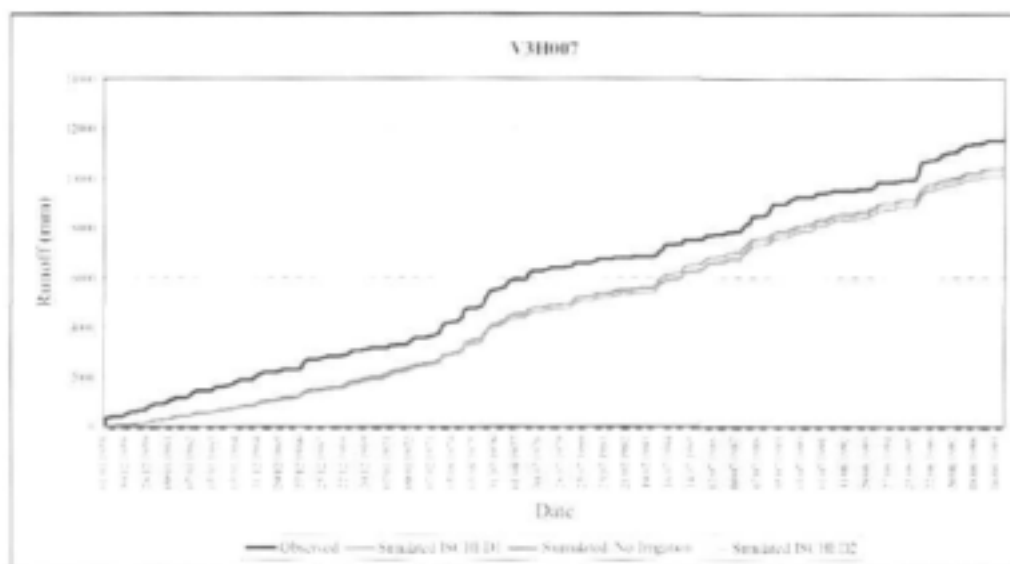


Figure 4.93 Accumulated simulated and observed runoff at Gauging Weir V3H007

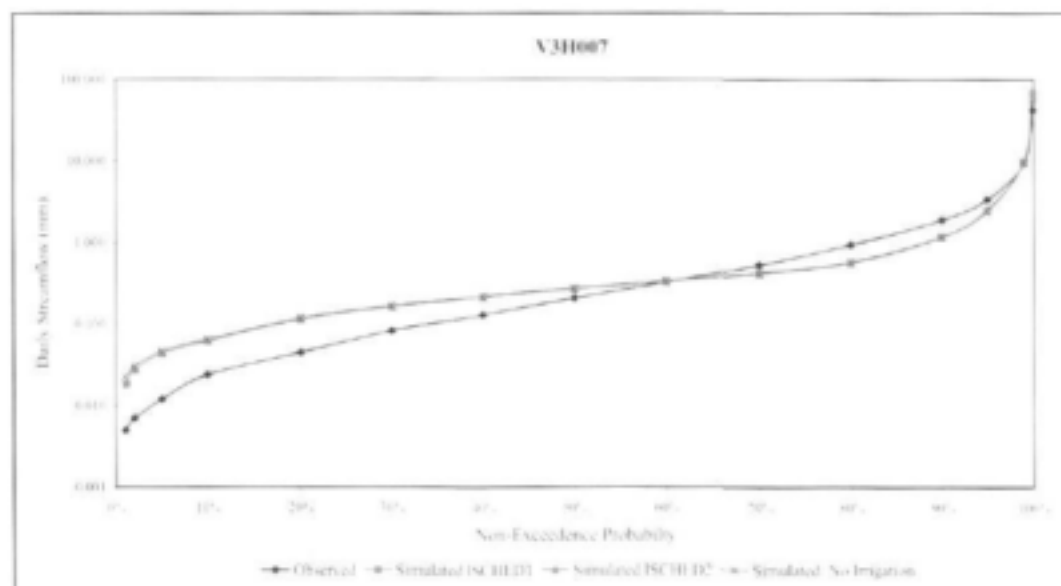


Figure 4.94 Frequency analysis of simulated and observed daily runoff depth at Gauging Weir V3H007

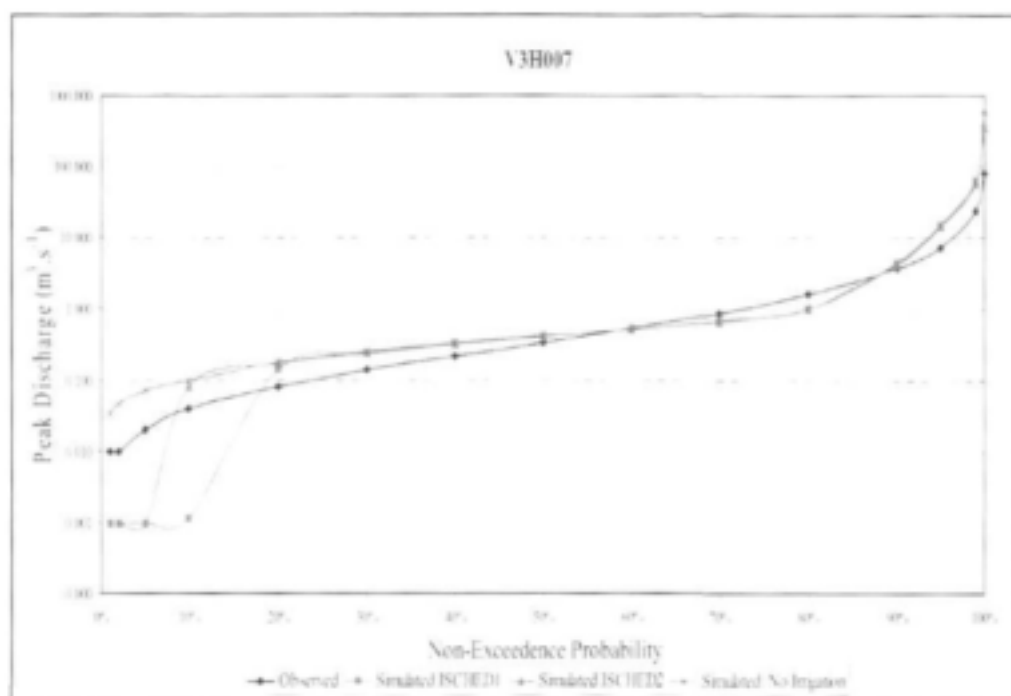


Figure 4.95 Frequency analysis of simulated and observed daily peak discharges at Gauging Weir V3H007

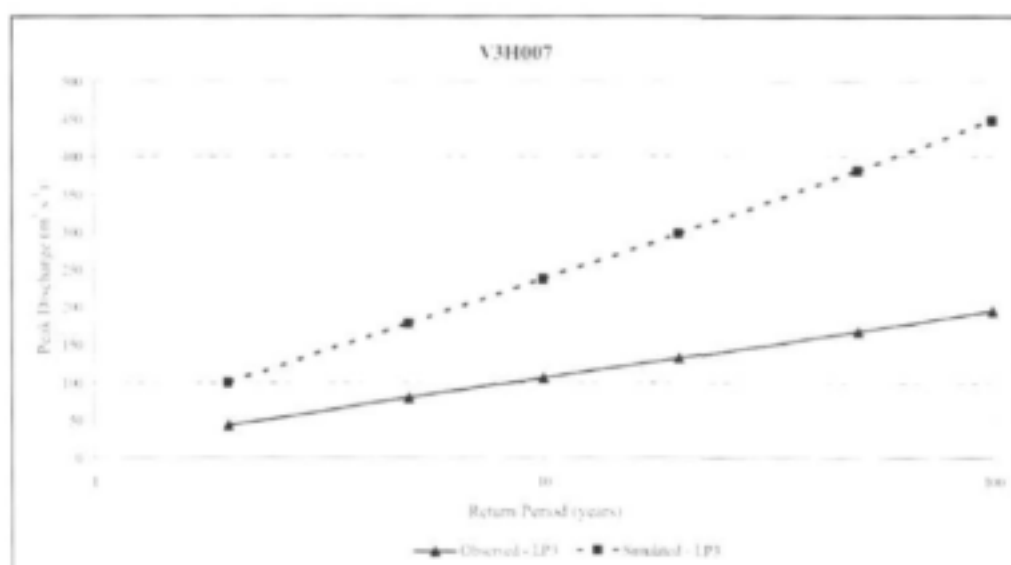


Figure 4.96 Design floods computed from the observed and simulated data at V3H007

4.5.9. Gauging Weir V3H009

The accumulated simulated and observed runoff at V3H009 (155.0 km²) for the period of available reliable observed flow data is shown in Figure 4.97 and a frequency analysis of daily runoff depth is shown in Figure 4.98. These results indicate that the CSM, with irrigation simulated, under-simulated the observed runoff volume until approximately 1980, after which the simulated and observed accumulated volumes increase at approximately the same rate. It is postulated that the levels of irrigation in the catchment increased up to approximately 1980 and have remained more or less static from then on. This could explain the improved simulations which include irrigation (after 1985) and the good simulation which exclude irrigation (prior to 1985).

As shown in Figure 4.99, the frequency distribution of daily peak discharge performed on the simulated and observed data are similar, particularly for the larger floods. The design floods computed from the observed and simulated data for various periods are shown in Figure 4.100 and the values computed from the observed data exceed those computed from the simulated values, particularly for higher return periods.

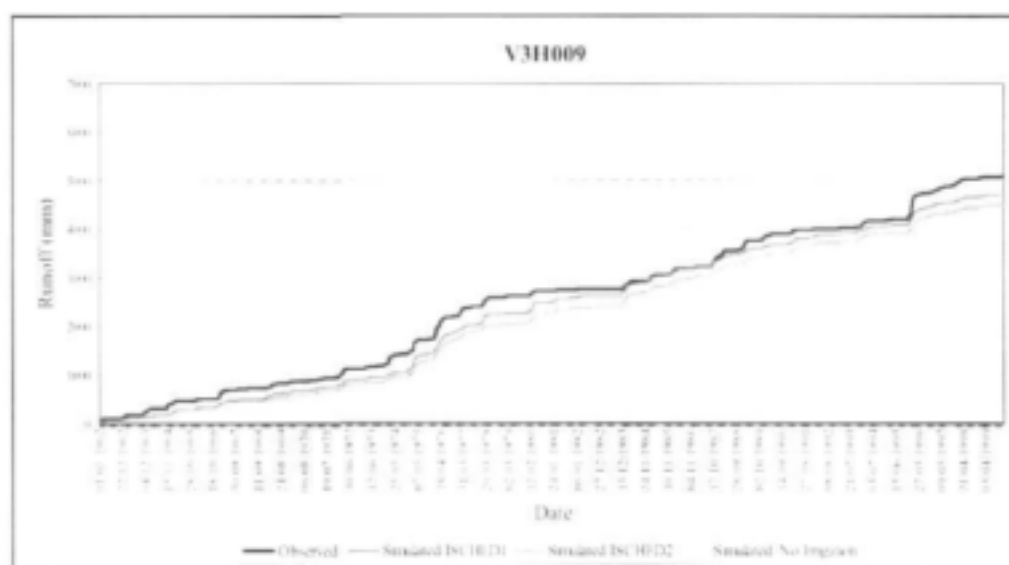


Figure 4.97 Accumulated simulated and observed runoff at Gauging Weir V3H009

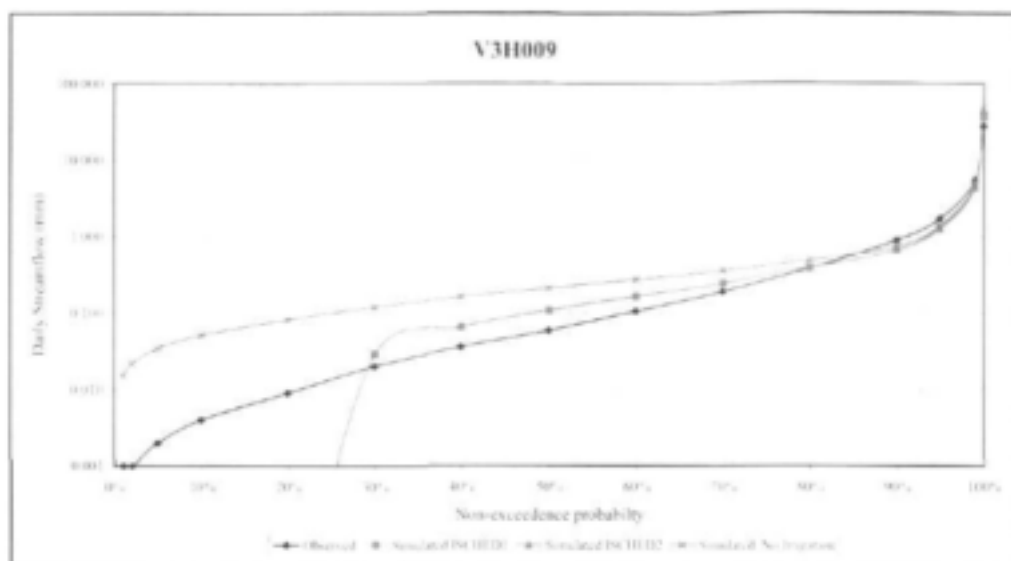


Figure 4.98 Frequency analysis of simulated and observed daily runoff depth at Gauging Weir V3H009

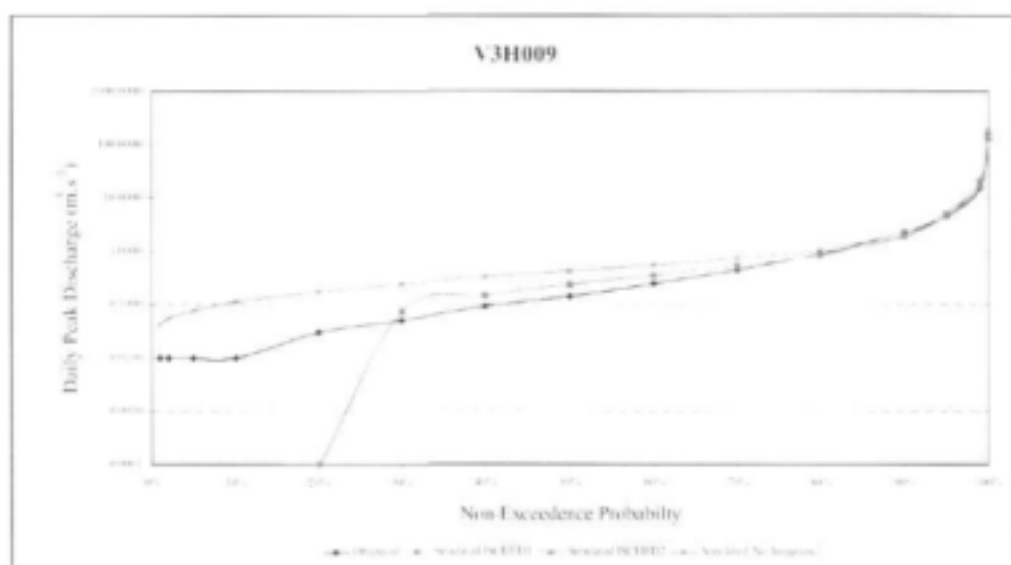


Figure 4.99 Frequency analysis of simulated and observed daily peak discharges at Gauging Weir V3H009

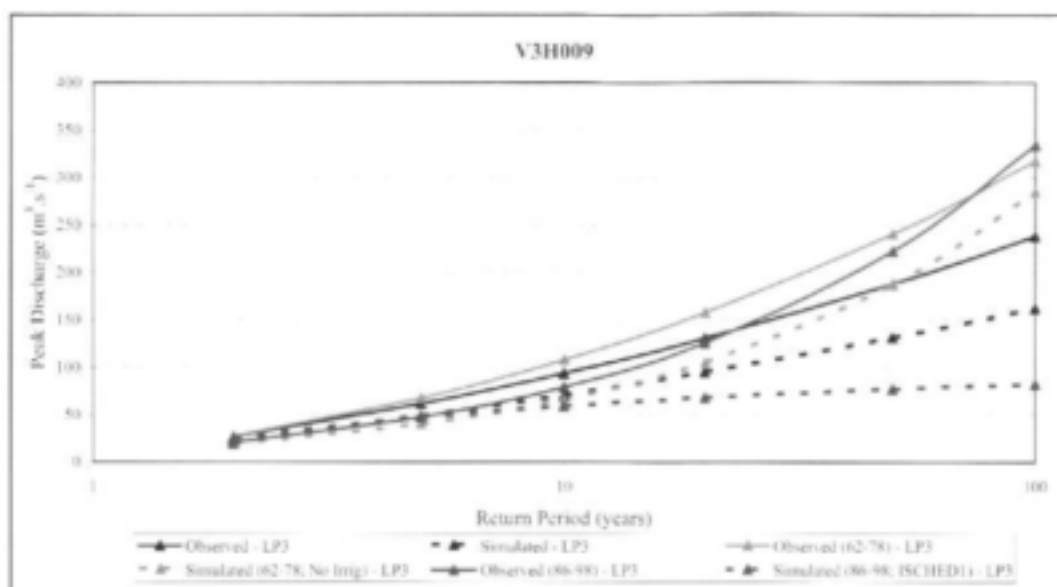


Figure 4.100 Design floods computed from the observed and simulated data at V3H009

4.5.10. Gauging Weir V6H004

The accumulated simulated and observed runoff at V6H004 (654.5 km²) for the period of available reliable observed flow data is shown in Figure 4.101 and a frequency analysis of daily runoff depth is shown in Figure 4.102. These results indicate that the CSM slightly over-simulated the observed runoff volume up to approximately 1984. This could have been caused by the 1999 land cover information used in the simulation. The distribution of the observed and simulated runoff volumes is similar, particularly for larger events.

As shown in Figure 4.103, the frequency distribution of daily peak discharges performed on the simulated and observed values indicates that the larger peaks are over-simulated. The design floods computed from the observed and simulated data are shown in Figure 4.104 and include design peak discharges estimated using a combination of values from the primary flow data and from van Bladeren (2000), termed "Primary and VB", and only from the primary data. The trends in the design peak discharge follows the trends in simulated peak discharge and the simulated values exceed the values computed from the observed data.

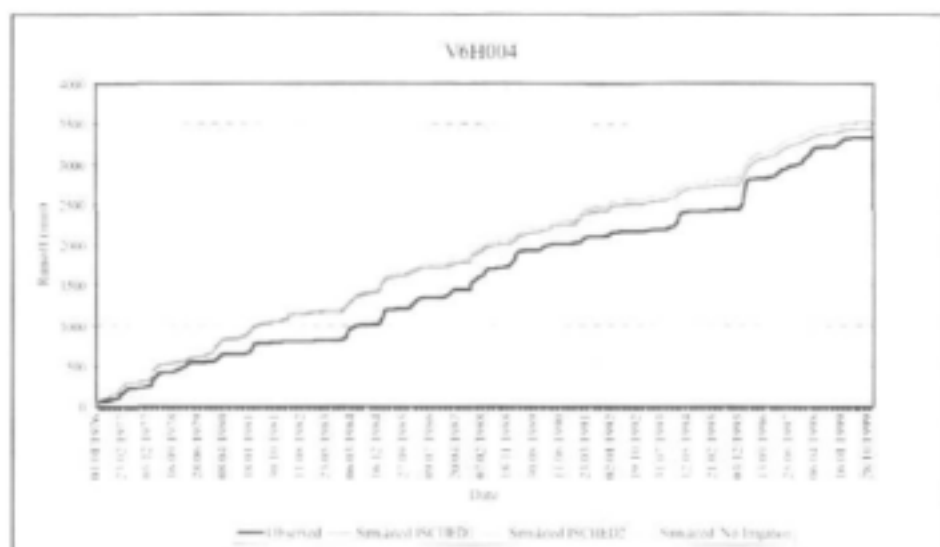


Figure 4.101 Accumulated simulated and observed runoff at Gauging Weir V6H004

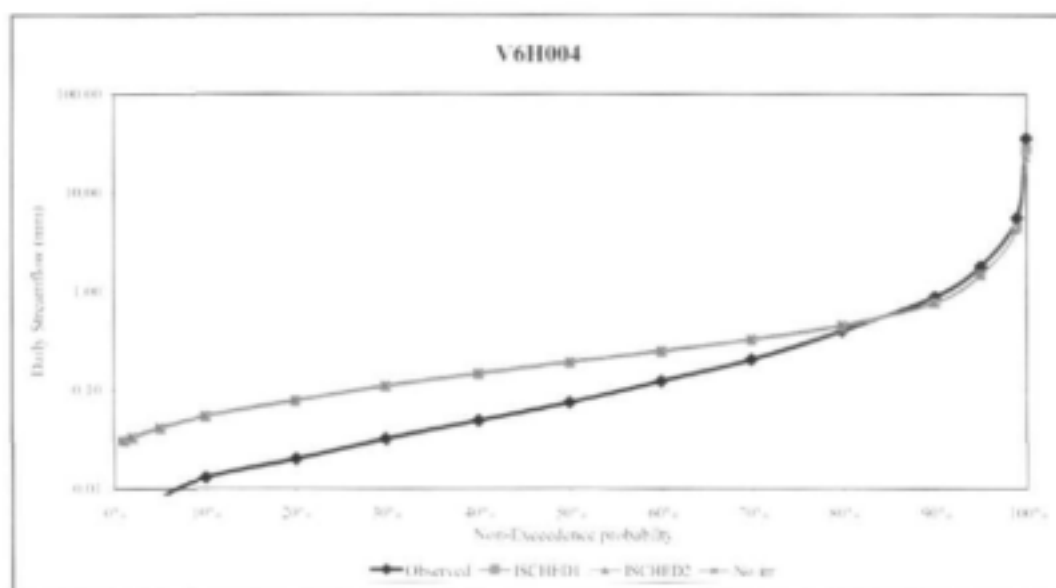


Figure 4.102 Frequency analysis of simulated and observed daily runoff depth at Gauging Weir V6H004

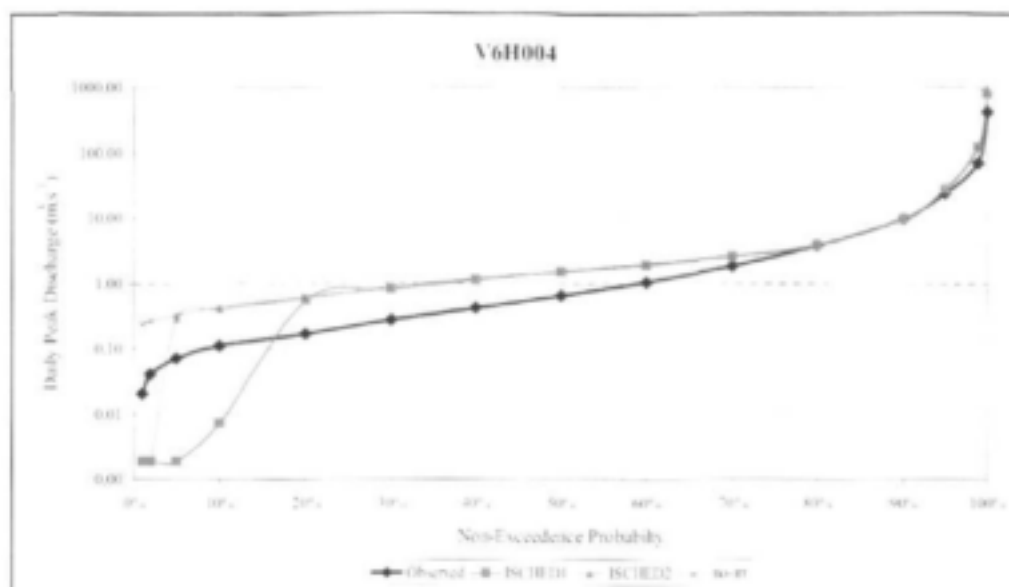


Figure 4.103 Frequency analysis of simulated and observed daily peak discharges at Gauging Weir V6H004

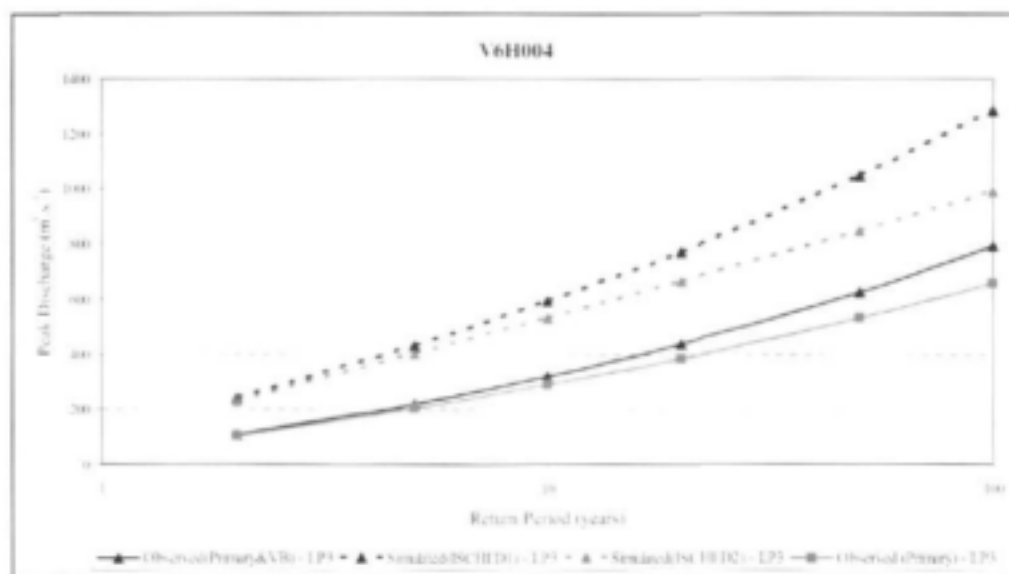


Figure 4.104 Design floods computed from the observed and simulated data at V6H004

The results from the verifications indicate the complexities involved in the modelling of an operational catchment where the observed data are not perfect, transfers of water between catchments occur, irrigation is developed over time and other land cover changes probably also take place. The problems associated with estimating rainfall over a catchment using a single raingauge are also evident. However, despite the above issues, the distribution of simulated volumes generally compared well with the observed distribution, over a range of catchment sizes. The distribution of the simulated peak discharges generally compared well with the observed distribution for the smaller catchments ($< 150 \text{ km}^2$), but were usually not as good for larger

catchments. The results for the design peak discharges reflect the results for the distribution of peak discharges. These results generated in this study are discussed in the following chapter.

5. DISCUSSION AND CONCLUSIONS

The estimation of design floods is necessary for the design of hydraulic structures. The under estimation of design floods will result in the failure of hydraulic structures with consequent economic losses and possible loss of life. The over design of hydraulic structures results in inefficient use of resources.

In South Africa, although not legislated, a number of methods for the estimation of design floods are commonly used. Where adequate streamflow data are available to perform a frequency analysis on the observed data, the LP3 probability distribution is recommended for use, although the GEV distribution has also been used to estimate design floods. In the absence of adequate streamflow data, a number of rainfall event-based approaches are commonly used to estimate design floods. These include the Rational, unit hydrograph and SCS-SA methods. The procedures for applying the widely used unit hydrograph approach in South Africa were developed in the early 1970s and very little development or refinement has been made since then. Major shortcomings of most event-based methods include the implicit assumption that the exceedance probability of the simulated streamflow is the same as that of the input design rainfall, and that antecedent soil moisture conditions prior to large storm events are not accounted for. The developments in the SCS-SA method include the joint probability of rainfall and runoff and also accounts for regionalised, typical antecedent soil moisture conditions prior to large rainfall events. The limitations of event-based approaches to design flood estimation can be overcome by adopting a continuous simulation approach to rainfall-runoff modelling.

Internationally, the use of continuous simulation modelling (CSM) to estimate design floods is gathering momentum given the improvements in modelling, data sets and advantages of such an approach. These include modelling the major processes responsible for converting the input catchment rainfall into runoff, the use of a continuous water budget model for the catchment so that antecedent conditions prior to each rainfall event are known, and computing the exceedance probability directly from the simulated streamflow. The objectives of this project were to develop a methodology, using a continuous simulation modelling approach, which could be used for flood frequency estimation in South Africa.

The *ACRU* model was selected for use in this study as it operates on daily time step, the simulated streamflow is sensitive to soils and land cover and the model has been used to estimate design floods in previous pilot studies. A number of areas were identified where the *ACRU* modelling system could be further developed or refined for design flood estimation. These included:

- An investigation into the scale of application and levels of soils and land cover information required to apply the *ACRU* model for continuous simulation modelling in order to estimate design floods.
- The development and evaluation of a method to disaggregate daily rainfall into hourly totals in South Africa in order to improve the shape of simulated hydrographs and peak discharge.
- An assessment of procedures used to merge raingauge and radar data and the use of the merged rainfall fields in the development of a methodology to improve the estimation of rainfall over a catchment when using historical gauged daily rainfall data.

- An assessment of the stochastic, fine resolution space-time String-of-Beads rainfall model to simulate long series of rainfall over a catchment which would enable long series of simulated streamflow to be generated in order to estimate design floods.
- The development and assessment of techniques for flood routing in ungauged catchments.
- In a parallel study, the development and application of adjustments to Curve Numbers for antecedent soil moisture conditions, used in the SCS design flood estimation model, was investigated.

These developments and refinements to the *ACRU* modelling system made in this study were incorporated into the model and the Thukela Catchment was used to assess the use of continuous simulation modelling for design flood estimation.

5.1. Scales Issues Related to the Configuration of the *ACRU* Model for Design Flood Estimation

In the development of a CSM system in this study, the spatial resolution at which continuous simulation modelling is implemented is important in order to model the hydrological responses from heterogeneous catchments. It was therefore necessary to investigate the appropriate range of scales at which the CSM system should be applied to identify the appropriate levels of representation of input information. Soil and land cover information play an important role in a CSM as these are the prime regulators of a catchments response to rainfall, and therefore directly influence the hydrological response of a catchment. The objective of this component of the study was to investigate the appropriate scale at which the CSM should be configured with respect to levels of spatial disaggregation of a catchment and of soil and land cover information required to give optimum results i.e. to investigate the optimal level of catchment discretisation for the *ACRU* model used in this study. Simulations at quaternary catchment (QC) scale and sub-quaternary scale (sub-QC) with different levels of soils and land cover information were undertaken in the Thukela Catchment.

The methodology adopted to perform this investigation required relatively un-impacted upstream catchments which had adequate and reliable data. This became a limitation to the study as not many catchments were un-impacted and which had long periods of record. Three QCs were selected with catchment areas ranging from 129 to 544 km². The majority of QCs within the Thukela catchment have catchment areas less than 500 km² and it was thus assumed that the results obtained for the selected QCs would be valid for all QCs in the Thukela catchment. The results detailed in Section 3.2 indicate that:

- The larger QCs should not be modelled as lumped entities and should be discretised to hydrological response units (HRUs) as the best simulations were obtained using HRU scenarios.
- The use of area weighted, and not modal soils information, resulted in the best simulations.
- Modelling using more than one driver rainfall station per sub-QC yielded better results in QC 59 than when a single driver rainfall station was used, but the use of additional rainfall stations in QC 72 did not make much difference to simulated results. This could be attributed to the fact that there were not many representative rainfall stations in QC 72 which had reliable data.

The recommendations from these results are that the optimum level of catchment discretisation in the Thukela catchment should include sub-quaternary catchments where appropriate, and all sub-catchments should be modelled using HRUs with area weighted soils information and, where possible, more than one driver rainfall station per QC should be used to represent the rainfall in the sub-catchments which constitute the QC.

5.2. Temporal Disaggregation of Daily Rainfall

The temporal distribution of rainfall, viz. the distribution of rainfall intensity during a storm, is an important factor affecting the timing and magnitude of peak flow from a catchment and hence the flood-generating potential of rainfall events. Rainfall disaggregation refers to producing high temporal resolution rainfall values which can be aggregated to give values equal to observed coarser-scale totals.

Prior to this study, the *ACRU* model utilised four fixed regionalised synthetic temporal distributions to disaggregate daily rainfall. This suggests that a single distribution can be used to represent the temporal distribution of rainfall for a particular region. This, however, is not realistic and analysis of rainfall data shows that at a given location there are several temporal patterns ranging from nearly uniform rainfall to highly variable rainfall. Furthermore, the peak intensity can occur during any hour of the day, adding to the variability of temporal rainfall patterns. In order to account for the variability of temporal patterns of rainfall, a regionalised stochastic approach to daily rainfall disaggregation was developed in this study, as described in Section 3.3.

The daily rainfall disaggregation model developed is capable of producing 480 different temporal patterns with ranging levels of uniformity. The methodology was assessed independently at 15 sites in South Africa and both moments, statistics and extreme rainfall events were analysed. The results indicate that the disaggregation model reproduced the general distribution of rainfall well, both when observed short duration rainfall data are available as well when short duration rainfall data are not available, but that the structure (sequencing) of the disaggregated rainfall requires additional refinements.

5.3. Spatial Distribution of Rainfall

Rainfall is highly variable in space and time and is the major driving hydrological force. Hence it is crucial to obtain an accurate estimate of rainfall in a catchment when modelling the hydrological response from a catchment. Raingauges measure rainfall directly and rainfall depth accumulated over the period of interest is measured with a high degree of accuracy at points where the gauges are located. However, raingauge networks are too sparse to capture the spatial variability of rainfall. Radar, on the other hand, measures a volume-averaged reflected signal power which is used to estimate rainfall and captures the spatial distribution of rainfall but needs to be calibrated using gauged rainfall data. Merged rainfall fields, derived from radar and raingauges, are currently the best estimate of the "true" rainfall field.

The objective of this component of the project was to improve the estimation of real and stochastic rainfall in a catchment. This was achieved by assessing a methodology to merge raingauge and radar data. Based on the premise that the merged rainfall fields are the best estimate of catchment rainfall, the short periods of available radar data were used to develop relationships between the merged rainfall fields for a

catchment and data from a selected raingauge, which could then be used to adjust the much longer record of historical gauged rainfall data to better represent rainfall in the catchment. The String-of-Beads-Model (SBM), which is a detailed space-time stochastic rainfall model, was also assessed for application to generate long sequences of rainfall over a catchment for use in a CSM. The Liebenbergsvlei Catchment was used as a study site as both radar and rainfall data from a dense network of gauges were available.

The merging technique was validated against data from tipping bucket raingauges used in the conditioning of the radar images. The conditional merging technique is intended to retain the rainfall depths used in the conditioning of the radar images in the merged rainfall field. The results obtained indicated that gauged rainfalls at the conditioning raingauges were not always retained, as the merging technique developed by Sinclair (2004) masks the area where the radar did not register any rainfall, even though raingauges in this area may have reported rainfall. This was done to avoid false rainfall in other parts of the area. When the masked values were removed from the comparison, a nearly perfect relationship was obtained between the conditioning raingauge data and the merged pixel rainfall values located at the conditioning raingauge. With the exception of the masked rainfall, the merging algorithm was determined to be correct.

The merging technique was independently verified using daily raingauges which were not used in the conditioning of the radar images. For most sub-catchments reasonably good verifications were obtained. Reasonably good relationships were generally found between the merged rainfall fields and selected raingauge data. It was noted that the raingauges selected to represent the areal rainfall of the sub-catchments generally over-estimated the mean areal merged rainfall values of the sub-catchments by between 5% and 50%. The methodology developed can be used to improve estimates of average historical catchment rainfall for use in modelling and other hydrological studies, until such time as the period of record of the merged rainfall fields is adequate for hydrological studies.

The assessment of the SBM indicates that the model reproduced the observed statistics at a daily time scale reasonably well and better than at monthly or annual time scales. This result is not unexpected as the SBM is a short duration rainfall model designed to mimic rainfall values at a detailed temporal and spatial resolution and small errors at 5 minutes durations accumulate over longer durations to the errors evident at the daily and longer time scales. Spatially, the SBM reproduced the statistics of the selected raingauges used in the study. Therefore, it is concluded that an appropriately calibrated SBM may be used in rainfall-runoff modelling which requires rainfall at detailed spatial and temporal resolutions and could be used as input to a CSM system in order to estimated design floods.

5.4. Flood Routing in Ungauged River Reaches

Hydrographs are lagged and attenuated as they flow down river reaches and through dams. In a CSM it is necessary to model the changes to hydrographs as they are routed through a catchment.

The Muskingum-Cunge method for flood routing is implemented in the *ACRU* model. In ungauged catchments, the user is required to specify the cross-sectional shape and dimensions of the reach, in addition to the slope and roughness coefficient for the reach. This input information is then used to estimate the depth : discharge

relationship for the reach. In this study it was shown that the computed outflow hydrographs generated using the Muskingum-Cunge method, both with empirically estimated variables and variables estimated from cross-sections of the selected rivers, resulted in reasonably accurate computed outflow hydrographs with respect to peak discharge, timing of peak flow and volume. Hence, it is concluded that the Muskingum-Cunge method, with variables estimated using empirical relationships, can be applied to route floods in ungauged catchments in the Thukela Catchment and it is postulated that the method can be used to route floods in other ungauged rivers in South Africa.

5.5. Application of a Climate Classification System for Regional Adjustments of SCS Curve Numbers

The SCS method is a design event based approach to design flood estimation and has been adapted for use in South Africa. The refinements to the SCS method for South Africa include the development of techniques to account for typical soil moisture status prior to large storms. Stormflow response is highly sensitive to a catchment's wetness. Many researchers have proposed that procedures to adjust Curve Numbers (CNs) according to a soil water budget provide realistic estimates of stormflow and peak discharge than when using only accumulated antecedent rainfall depths. However, the direct application of soil water budgeting procedures requires long and accurate daily rainfall and temperature records, in order to estimate the change in soil moisture storage, ΔS , prior to the runoff producing storm events. The problem then arises of how to estimate the values of ΔS in regions where only limited hydrological and very basic monthly climatological information is available.

The objective of this component of the study was to approach the soil water status of a catchment as a climatologically driven variable by assuming that changes to CNs due to Antecedent Soil Moisture (ASM) are similar internationally for similar climatic regions, and that climatic regions may be represented by a standard climate classification, such as the Köppen classification system, which was selected in this study. It was further hypothesised that within a specified Köppen Climate Class (KCC), ΔS was likely to be a function of the distribution of Mean Annual Precipitation (MAP) for major soil/vegetation combinations.

The concept of estimating ΔS from MAP for each KCC and specified soil/vegetation combinations was termed the ACRU-Köppen method. The approach is conceptually sound, computationally simple, complies with hydrological fundamentals and does not move beyond the original intent of the SCS technique. A major strength of the approach is that it is a relatively simple technique, does not require a high level of expertise and is not data demanding for application. The use of the ACRU-Köppen method of CN adjustment was found to work well in a catchment in Eritrea and it is postulated that the method could be used in other data sparse catchments for the same KCC and similar soil/land cover characteristics to improve modelling of stormflow volumes and peak discharges from small catchments in developing regions, where adequate and accurate hydro-meteorological information are often not available.

5.6. Application of the CSM in the Thukela Catchment

The above developments and refinements were incorporated into the ACRU model and the Thukela Catchment was used as a case study. The application of the model proved challenging in an operational catchment where the observed data are not

perfect, the network of daily raingauges is relatively sparse, transfers of water between catchments take place, irrigation is developed over time and other land cover changes occur.

Some challenges to modelling and design flood estimation and issues which were noted during the project include the following:

- The problems associated with estimating rainfall over a catchment using a single raingauge were evident in some results when the simulated and observed streamflow did not correspond for some events. However, even though there may not be a one-to-one correspondence between simulated and observed events, it is necessary for design flood estimation that the distribution of the larger events is similar.
- A thorough investigation into the reliability of the observed data indicated that the data from many flow gauging stations were not suitable for verification studies. The importance of using a physically-based, conceptual model is highlighted by the unreliability of the flow data from a number of the gauging weirs. The direct calibration of a model to these data would result in erroneous simulations.
- The stages recorded at many of the flow gauges exceeded their rating tables during larger events, which are the focus of this study. The moderate extension of the rating tables performed in this study is believed to add to the uncertainty of the accuracy of the observed flow data.
- The annual maximum series (AMS) extracted from the primary flow data generated using the extended rating tables generally corresponded well with AMS supplied by Van Bladeren (2000), although in some cases the AMS from Van Bladeren exceeded the extracted values.
- No detailed information which could be used in a daily time step model was obtained on the transfers of water between catchments, which limited the number of catchments which could be used for verification of the simulations.
- The areas irrigated in the sub-catchments were derived from the WARMS database. Without detailed information on irrigation management practices, two irrigation schedules were simulated and the results obtained indicate that the simulated volumes are not sensitive to different, but realistic, irrigation scheduling. The peak discharges obtained from the two irrigation schedules simulated were different as a result of the stochastic disaggregation of the daily rainfall.
- The development of irrigation in some sub-catchments impacted significantly on the streamflow simulated. This was evident in the results from a number of catchments by good correspondence between accumulated observed and simulated runoff in the early part of the record when irrigation was not simulated, and in the latter part when irrigation was simulated.
- The use of a static land cover in the simulations, which does not reflect changes over time of land cover in a catchment, resulted in better simulations closer to the date of when the land cover information used in the simulations was captured.

Despite the above issues, the distribution of simulated volumes generally compared well with the distribution of the observed data, over a range of catchment sizes. The distribution of the simulated peak discharges generally compared well with the observed distribution for the smaller catchments ($< 150 \text{ km}^2$), but were usually not as

good for larger catchments. The results for the simulated design peak discharges reflect the results for the distribution of peak discharges, but are deemed to be reliable and consistent enough for use in practice.

5.7. Conclusions

A number of conclusions regarding the results from this study with respect to the application of the *ACRU* modelling system for use in design flood estimation can be made. These include:

- In order to realistically model the hydrology of the larger quaternary catchments at a daily time step, it is necessary to delineate the quaternary catchment into sub-catchments and, where appropriate, model each sub-catchment as a number of HRUs.
- The use of area weighted, and not modal, soils information is recommended.
- The accurate estimation of catchment rainfall is crucial to modelling the correct hydrological responses and the merging of raingauge and radar data is the best estimate of the true rainfall field.
- The use of historical rainfall from raingauges to estimate catchment rainfall can be improved by using relationships developed between the short record of merged rainfall fields and corresponding data from a selected raingauge.
- The SBM model holds much potential to generate detailed space and time stochastic rainfall values which can be used as input to a CSM system.
- The method of disaggregating daily rainfall developed in this project introduces a stochastic element into the *ACRU* model and the disaggregation procedures can be used by other hydrological studies where the disaggregation of daily rainfall is required.
- The use of the *ACRU* model for design flood estimation was shown to produce distributions of simulated daily volumes which generally reflected the observed distribution in catchments for a range of catchment areas.
- The distribution of the daily simulated peak discharges generally reflected the observed distribution in smaller catchments ($< 150 \text{ km}^2$). In larger catchments the good simulation of the distribution of streamflow volumes did not consistently translate into good distributions of daily peak discharge.
- The advantage of continuous simulation modelling to provide consistent and reliable estimates of design floods have been highlighted in this study. This approach avoids calibration against observed flow data which may be erroneous and is able to reflect current and future conditions in the catchments in the estimation of design floods and thus avoid any non-stationarity which may be present in the observed flow data.
- This study has emphasised the need to improve the estimation of historical rainfall, irrigation and land cover changes in a catchment in order to realistically simulate observed hydrological responses.
- The difficulty in translating a flood volume into a realistic hydrograph with the correct peak, particularly on larger catchments, is evident from the results obtained.

The results from this study have shown that the use of the *ACRU* model as a CSM can simulate the hydrological responses from an operational catchment, despite the challenges related to data and operations in the catchment. The output from the CSM

has been shown to produce reasonable and consistent estimates of design floods, particularly in smaller catchments.

5.8. Recommendations for Future Research

In order to refine and improve the CSM for design flood estimation, the following recommendations are made:

- The method of adjusting historical raingauge data to estimate catchment rainfall should be applied for all catchments where radar data are available.
- The use of the merged rainfall fields created from the radar and raingauge data should be used as input to hydrological models where possible.
- In order to simulate historical flow data, the use of a time series of land cover information should be generated where possible and a database of changes to land cover should be captured and maintained for future use.
- An analysis of the hydrographs generated by the CSM should be made to further assess the performance of the model for design flood estimation.
- The translation of the runoff volume into a hydrograph and associated peak discharge requires further refinement. This may involve investigating the estimation of catchment lag and further investigation into the performance of flood routing algorithms for application in ungauged catchments.
- A more detailed study is required to assess the significance on the simulated performance of the stochastic disaggregation of daily rainfall.
- The SBM should be refined to be user friendly and easily calibrated against raingauge data and the SBM should be used to generate long sequences of streamflow for design flood estimation.
- The methodology for the temporal disaggregation of daily rainfall requires further refinement. It is recommended that the methodology used be applied on discrete ranges of daily rainfall i.e. for smaller and larger events. It is further recommended sequencing of the disaggregated hourly rainfalls be refined in order to improve the simulation of the structure of the rainfall, as measured by the lag autocorrelations, number of events and event durations.

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This project reviewed and evaluated the footprint achieved by the SANCOLD Guidelines. The researchers worked towards updating these guidelines in terms of international best practice. In the project, flood hydrograph-related information contained in the streamflow records of the last three decades were also brought into the project to update the data used in the old South African design flood practices. The other project objectives which were achieved in the research included:

- To establish updated Guidelines for the safety evaluation of dams in relation to floods
- To derive a methodology for design flood hydrograph estimation based on joint occurrence of flood peaks and flood volumes, through analysis of historically measured flood hydrographs in all regions of South Africa
- To develop a modernised set of design tools for the generation of complete flood hydrographs for dam safety evaluation or spillway design.

The project was carried out in 4 phases.

The first phase was an assessment of Local and International Practices Regarding Dam Safety in Relation to Floods. This phase was followed by data collection and the Improvement of Flood Hydrograph Generation Techniques for South Africa for Dam Safety Purposes. Phase 3 involved the development of a "Design Flood Hydrograph Toolbox", the purpose of which is to support the various components of dam safety evaluation in relation to floods. The final project output, which is reported in a separate report was a review of the SANCOLD Guidelines on Dam Safety in Relation to Floods. This report also included insights into specific aspects of the existing SANCOLD guidelines as well as recommendations for updating these guidelines or replacing them.

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